# SEISMIC DESIGN OF BRIDGE STRUCTURES WITH ALLOWANCE FOR LARGE RELATIVE GIRDER MOVEMENTS TO AVOID POUNDING

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**SUMMARY**

Pounding between bridge girders have been observed in almost all previous major earthquakes. This is because the gap size of conventional bridge expansion joint is usually only a few centimetres, which is not sufficient to preclude poundings owing to large relative displacements between bridge girders caused by the effect of varying vibration properties of adjacent bridge spans, varying ground motions at bridge supports and varying soil-structure interaction (SSI). In this work a new design of bridge expansion joint is introduced. Instead of tolerating pounding and providing possible mitigating measures, this new design approach enables large movement between bridge girders which makes a complete pounding preclusion possible. The new expansion joint is called Modular Expansion Joint (MEJ). The large movability is achieved by installing a number of small gaps in the joint. In this study, the MEJ gap size required to completely avoid girder pounding is investigated. The most significant influence factors -the varying vibration properties of adjacent bridge spans, the effect of SSI and ground motion spatial variation on expansion joint size required to preclude pounding- are calculated. Discussions on the relative importance of various structural and ground motion properties in generating relative displacements of adjacent bridge girders are made.

## INTRODUCTION

The response of structures to earthquake motions depends on dynamic properties of structures, site conditions and earthquake ground motions. In many analyses of structural responses to earthquake excitations, only dynamic properties of the structures are modelled in detail. The effects of site conditions, which influence the soil-structure interaction, are ignored by assuming structures sitting on a rigid ground. As for earthquake ground motion, usually only its amplitude, frequency content and duration are modelled in either spectral analysis or time history analysis. Earthquake ground motion spatial variation at different structural supports, which is inevitable owing to seismic wave propagation, is often neglected. While in the investigation of the response of adjacent buildings an assumption of the same ground excitation is justifiable, for the determination of the response of long-extended structures, e.g. long gas transmission line or bridges, ground motion spatial variation owing to seismic wave propagation from one structural support to the others needs to be considered. Therefore, when analysing dynamic response of a bridge structure, not only the bridge properties, but also the ground motion spatial variations and possible unequal soil-structure interaction due to non-uniform soil and structural properties should be properly modelled [1].

Despite a large number of investigations and implementations in the design specifications to avoid or to mitigate the consequences of relative movements between adjacent bridge structures (spans), damages associated with relative movements of adjacent bridge spans such as pounding between bridge decks, and unseating of bridge girders or even collapses of bridge spans have still been observed in all previous major earthquakes, e.g. the Northridge earthquake in 1994 [2], the Kobe earthquake in 1995 [3], the Chi-Chi earthquake in 1999 [4] and the Yogyakarta earthquake in 2006 [5]. In the recent earthquake in Gisborne in December 2007 damages due to relative responses of adjacent buildings are also observed.

Many researchers investigated the effect of pounding on adjacent building structures (e.g. Anagnostopoulos [6]; Anagnostopoulos and Spliliopoulos [7] and Athanassiadou, [8]), estimated the separation distance required to prevent adjacent buildings from pounding (e.g. Penzien [9]; Hao and Liu [10]; Hao and Shen [11]), and proposed mitigation measures to reduce the pounding effect (e.g. Luco and Barros [12]). Hao and Zhang [13] investigated the possible influence of spatial ground motion on relative building responses. Most of the studies neglected the effect of the soil-structure interaction. Rahman et al. [14] included the soil effect in their studies. However, they considered only static soil stiffness, i.e. adding static soil springs in the numerical model. Only very

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few investigations had taken the dynamic soil stiffness into consideration (e.g. Chouw [15]). Pounding and unseating of bridge structures due to earthquake excitations have also been intensively studied. Although in the case of long bridges the influence of the spatial variation of the ground excitations can be significant, most studies were performed under the assumption of uniform ground motions. DesRoches and Muthumar [16], Ruangrassamee and Kawashima [17] and Zhu et al. [18], for example, considered measures to reduce the effect of pounding between adjacent bridge girders. An overview of possible mitigation measures is given by Yashinsky and Karshenas [19]. In these studies, the effect of ground motion spatial variation is either completely neglected, or only the time delay of spatial ground motions is included. The influence of spatially varying ground motions on pounding responses of bridge structures is performed by only a few researchers (e.g. Hao [20]; Zanardo *et al.* [21]). In their studies, often the bridges are assumed to be fixed at their base without considering the soil-structure interaction effect. When the interaction between bridge structures and subsoil is included, only frequency-independent soil stiffness is applied and ground motion spatial variation is not considered (e.g. Zhu *et al.* [22]; Tongaonkar and Jangid [23]). In this study frequency-dependent soil stiffness and the spatially varying ground motions are considered in the analysis.

In most previous studies of bridge pounding responses, attentions were paid to pounding effects on bridge decks and mitigation measures to reduce the pounding damage. Only very limited number of studies estimated the required separation between bridge decks to avoid pounding. This is because it is not possible to make the gap size in conventional bridge expansion joint large enough to completely avoid possible poundings during strong earthquake shaking owing to the functionality consideration of bridge expansion joint for smooth traffic flow. Recently, a new design of bridge expansion joint, namely the Modular Expansion Joint (MEJ), is under rapid development. A MEJ allows for large relative movement between bridge decks. It makes completely preclude pounding between bridge decks possible. Only one preliminary study has been conducted to investigate the required separation length for a MEJ to avoid pounding during earthquake shaking [24]. The primary differences between that study and the current work include:

1. In the previous study spatial ground motions considered are based on a near source ground motion attenuation model provided by Ambraseys and Douglas [25]. In the present study, design earthquake ground motions specified in Japanese seismic design code [26] are considered.
2. Besides the gap size of MEJ required to avoid pounding as estimated in previous study, the desired opening movability is also investigated in this study. This is essential for proper design of MEJ, because a MEJ should not only be able to preclude pounding during closing girder movements. It should also function properly without having any damage during expected large opening relative movements of bridge girders.
3. Not only one but three different fundamental periods of the adjacent bridge structure are considered to cover a larger range of possible cases.
4. While in the previous work only ground motions of one particular frequency content are considered, in this study ground motions with three different ranges of dominant frequencies, which represent three different soil conditions, are taken into account.
5. While in the previous work the simultaneous influence of wave passage and coherency loss is investigated, in current study the effect of commonly assumed time delay only is considered as well.

## BRIDGE GIRDER WITH MEJ ALLOWING FOR LARGE RELATIVE MOVEMENTS

### Bridge structures

In this study two bridge segments with the same height of 9 m are considered (Figure 4). The movement of the girder of each bridge segment is modelled with a single degree of freedom. It is assumed that each bridge segment with an assumed fixed base has a damping ratio of 5 %. The bridge segments with multiple bridge piers are described as single-pier bridge structures. It is assumed that each footing has the dimension of 9 m x 9 m and the soft subsoil is a homogeneous half space with the shear wave velocity $c_s$ of 100 m/s, the density $\rho$ of 2,000 kg/m$^3$ and the Poisson's ratio $\nu$ of 0.33. To limit the considered influence parameters it is assumed that the soil has no material damping. Hence, only the effect of the radiation damping owing to wave propagation from the vibrating footings is taken into account.

In the numerical analysis substructure technique in the Laplace domain is applied. The bridge structures with their footings and supporting subsoil are modelled by using finite elements and boundary elements, respectively. It is assumed that the bridge structures remain linear. Since no pounding between bridge girders should take place, only linear analysis is required.

It is assumed that each structural member has a continuous distribution of mass and stiffness along the member. After transforming the equation of motions into the Laplace domain the dynamic stiffness for transversal and longitudinal vibrations of a structural member is determined by solving the equation of motions analytically. The dynamic stiffness of each bridge segment $\left[\tilde{K}^b\right]$ is obtained by adding the dynamic stiffness of each member by using the direct stiffness approach. (˜) indicates a vector or matrix in the Laplace domain. Details of the derivation of the dynamic stiffness of the bridge structures are given in [27]. To consider the wave propagation due to interaction between subsoil and footings of bridge structures the wave equation is transformed into the Laplace domain. By applying the full-space fundamental solution and by assuming the distribution of displacement and traction along the soil surface, the relationship between traction and displacement is defined. An introduction of the boundary element areas leads to the dynamic stiffness $\left[\tilde{K}^s\right]$ of the soil. Details of the derivation of the dynamic soil stiffness are given in [28]. A coupling of the two subsystems, i.e., subsoil and bridge structures, leads to the governing equation of the whole system

$$\begin{bmatrix} \tilde{K}_{bb}^{bI} & \tilde{K}_{bc}^{bI} & 0 & 0 \\ \tilde{K}_{cb}^{bI} & \tilde{K}_{cc}^{bI}+\tilde{K}_{cc}^{sI} & 0 & \tilde{K}_{cc}^{sI\,II} \\ 0 & 0 & \tilde{K}_{bb}^{bII} & \tilde{K}_{bc}^{bII} \\ 0 & \tilde{K}_{cc}^{sII\,I} & \tilde{K}_{cb}^{bII} & \tilde{K}_{cc}^{bII}+\tilde{K}_{cc}^{sII} \end{bmatrix} \begin{Bmatrix} \tilde{u}_b^{bI} \\ \tilde{u}_c^{bI} \\ \tilde{u}_b^{bII} \\ \tilde{u}_c^{bII} \end{Bmatrix} = \begin{Bmatrix} \tilde{P}_b^{bI} \\ \tilde{P}_c^{bI} \\ \tilde{P}_b^{bII} \\ \tilde{P}_c^{bII} \end{Bmatrix} \tag{1}$$

The indices I and II stand for the left and right bridge structure with footing and subsoil, respectively. b, s and c represent the bridge, subsoil and the contact-degree-of-freedom at the interface between the footing and supporting ground, respectively. The terms $\tilde{K}_{cc}^{sII\,I}$ and $\tilde{K}_{cc}^{sI\,II}$ are the dynamic stiffness of the common subsoil of the whole system. For simplicity it is assumed that the interaction between the two adjacent bridges via common subsoil can be neglected. This substructure technique enables a correct consideration of frequency-dependent interaction between the bridge structures and their supporting ground. After transforming the load $\{P(t)\}$ into the Laplace domain by using equation (2), the response of the whole system can be obtained.

$$\{\tilde{P}(s)\} = \frac{1}{2\pi i}\int_0^\infty \{P(t)\}e^{-st}dt \tag{2}$$

where the Laplace parameter $s = \delta + i\omega$ and $i = \sqrt{-1}$ . The time history of the whole system can be determined by transforming the results back into the time domain

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

From the obtained response of the bridge structures the relative displacements of the neighbouring girders can then be determined.

**Spatially varying ground motions**

To investigate the influence of strong earthquakes on the bridge structures the design spectra for the ground motion type II according to the Japanese design specification for soft, medium and hard soil sites are considered [26]. The design spectra were introduced in 1996 after the Kobe earthquake. During the earthquake many ground motions were recorded within a distance of 100 km from the epicentre. The spectra were defined by simply enveloping the response spectrum values of the recorded ground motions. They have a clear dominant frequency range. Figure 1 shows the current Japanese design spectra. Their corresponding frequency contents are listed in Table 1. In this study, the ground motion time histories are simulated to be compatible with these design spectra.

To simulate the spatial variation of ground motions the empirical function for coherency loss is used [29]. It was based on recorded ground motions at a dense seismograph array SMART-1 [30]. The empirical coherency loss function is given as

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

where $d_{ij}^l$ and $d_{ij}^t$ in metres are the projected distances between locations $i$ and $j$ on the ground surface in the seismic wave propagation direction and its perpendicular direction, respectively. $\beta_1$ and $\beta_2$ are two constants, and $\alpha_1(f)$ and $\alpha_2(f)$ are two functions. They are defined as

$$\alpha_n(f) = \frac{a_n}{f} + b_n f + c_n, \quad n = 1, 2 \tag{5}$$

The parameters used in the simulation are: $\beta_1 = 1.109\ 10^{-4}$, $\beta_2 = 6.73\ 10^{-5}$, $a_1 = 3.583\ 10^{-3}$, $b_1 = -1.811\ 10^{-5}$, $c_1 = 1.177\ 10^{-4}$, $a_2 = 5.163\ 10^{-3}$, $b_2 = -7.583\ 10^{-6}$, and $c_2 = -1.905\ 10^{-4}$, which were obtained by fitting the recorded motions during Event 45 at the SMART-1 array to equation (4) [30].

It should be noted that the spatial ground motions recorded during Event 45 at the SMART-1 array are highly cross correlated. To study the ground motion spatial variation effects, in this study intermediately and weakly correlated ground motions are also simulated. Because the ground motion spatial variation is not well understood yet, without

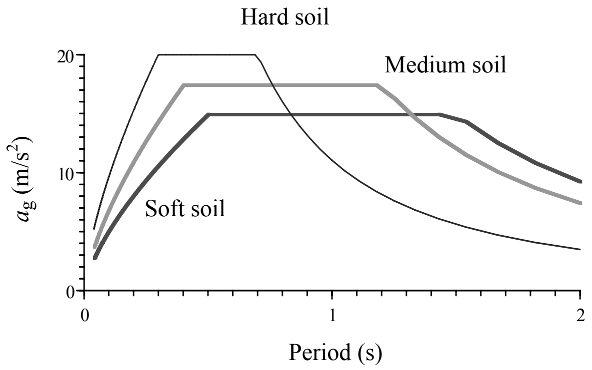


*Figure 1: Japanese design spectra.*

**Table 1. Dominant frequency range in design spectra of Japanese code.**

| Soil type | Dominant frequencies (Hz) | |
|---|---|---|
| Soft soil | 0.67 | 2.0 |
| Medium soil | 0.83 | 2.5 |
| Hard soil | 1.43 | 3.4 |

losing generality, the coefficients in the coherency loss function corresponding to the intermediately and weakly cross correlated spatial ground motions are modified from those derived from the recorded motions at the SMART-1 array. For intermediately correlated case the following parameters are used: $\beta_1 = 1.109/3\ 10^{-4}$, $\beta_2 = 6.73/3\ 10^{-5}$, $a_1 = 3.583/3\ 10^{-3}$, $a_2 = 5.163/3\ 10^{-3}$. For weakly correlated cases the parameters are: $\beta_1 = 1.109\ 10^{-4}$, $\beta_2 = 6.73\ 10^{-5}$, $a_1 = 3.583\ 10^{-2}$, $a_2 = 5.163\ 10^{-2}$. The coefficients $b_1$, $b_2$, and $c_1$, $c_2$ remain the same as for the highly correlated ground motions.

Figures 2(a)-(c) show one set of the spatially varying ground accelerations with a separation distance of 100 m along the wave propagation direction for soft, medium and hard soil conditions, respectively. The corresponding ground displacements are displayed in Figure 3. These ground motions are simulated according to the design response spectra given in Figure 1 and coherency loss function defined in equation (4). It should be noted that they well match the respective design spectrum individually and match the corresponding coherency loss function between each other. More information regarding spatial ground motion simulation can be found in [29]. As shown in the simulated ground motion time histories, the increase of the dominant frequencies with the soil stiffness significantly affects the ground displacements. Although the ground accelerations have almost the same peak value of about 6 m/s$^2$ as defined in the design spectra, the ground displacements decrease with increasing stiffness of the soil. Another factor, that is significant for the response of adjacent structures, is the relative ground movement at the two considered support locations of the adjacent structures. While in the case of soft soil, the maximum relative ground displacement reaches the value of 1.3 m, for medium and hard soil conditions the maximum values reduce drastically to 0.55 m and 0.3 m, respectively. Consequently, the effect of spatial ground motions with low dominant frequencies on the relative response of adjacent structures is expected to be more significant. In other words, the lower the dominant frequencies the ground motions have, the more prominent is the spatial variations of ground displacement and therefore stronger effect of ground motion spatial variation on relative displacement responses of adjacent structures.

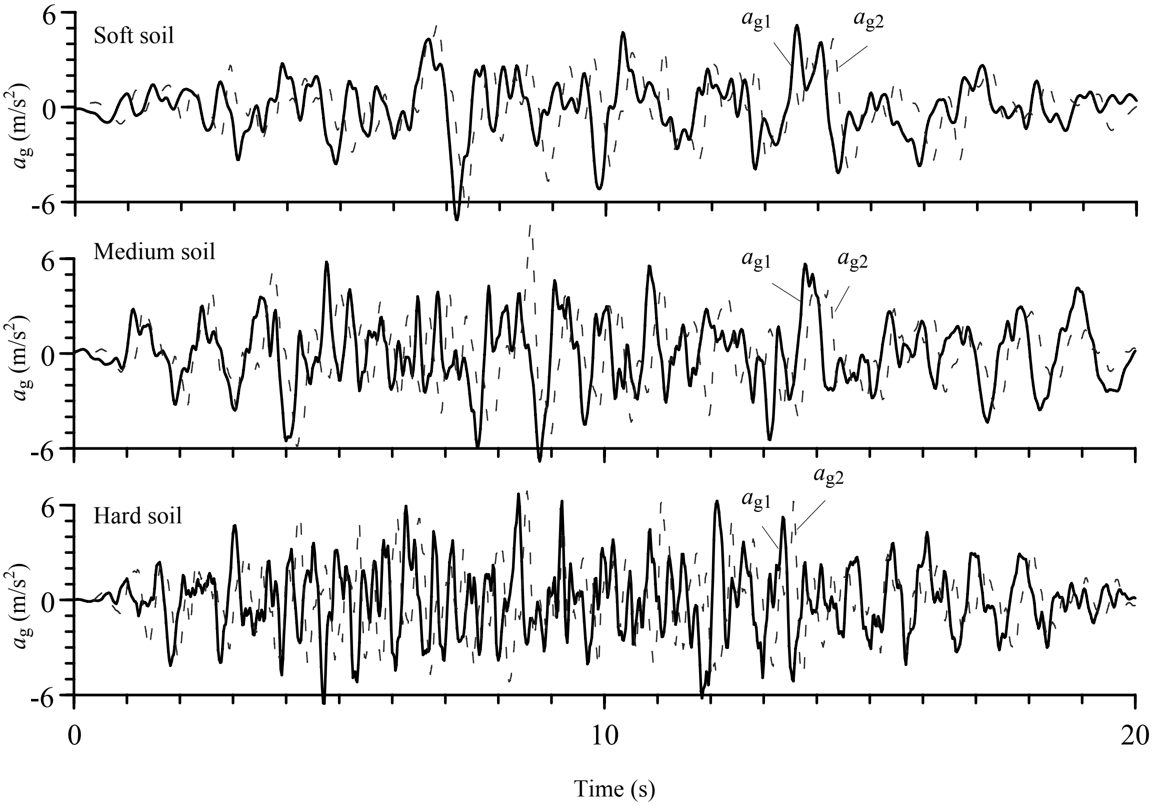


***Figure 2(a)-(c): Simulated intermediately correlated spatially varying ground accelerations $a_g(t)$ with $c_a$ = 500 m/s for (a) soft soil, (b) medium soil and (c) hard soil.***

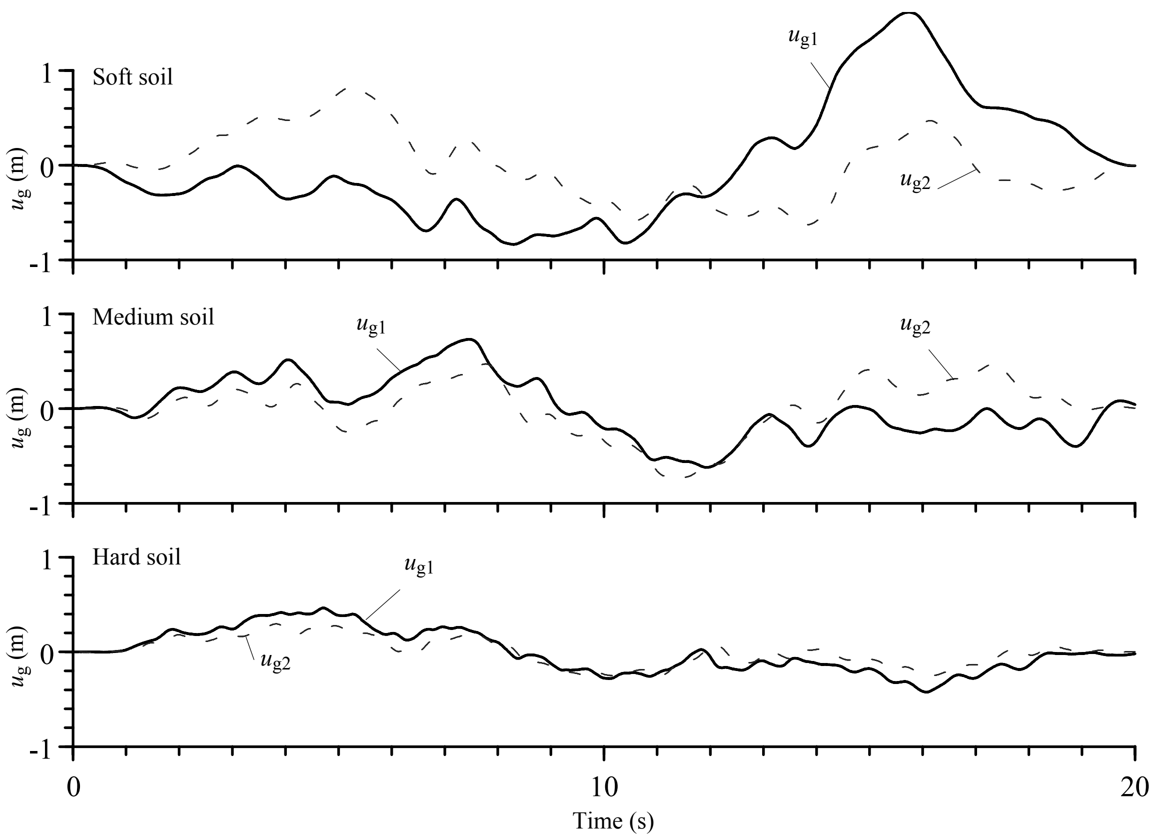


***Figure 3(a)-(c): Simulated intermediately correlated spatially varying ground displacements $u_g(t)$ with $c_a$ = 500 m/s for (a) soft soil, (b) medium soil and (c) hard soil.***

**A new design approach with MEJ**

Current design regulations (e.g. AASHTO [31]; CALTRANS, [32]; JRA [33]) recommend that neighbouring structures should have a sufficient distance to mitigate pounding damage. This allowable 'sufficient' distance is usually small because of the serviceability requirement as discussed above. It is achieved by adjusting the dynamic properties of neighbouring structures to make them have similar fundamental frequencies and hence in-phase overall vibrations. When the adjacent structures respond to the earthquake motions in phase, the relative movement between the structures is small. Consequently, pounding between structures might be avoided or its effect significantly reduced. This recommendation, however, relies mainly on the dynamic characteristics of the structures and ignores the effect of ground motion spatial variations and soil-structure interaction. When the ground excitations at adjacent structures vary strongly owing -for example- to changing local soil properties along the path of wave spreading, this recommendation can cause just adverse effect. Besides, changing soil properties at different bridge supports and unequal structural slenderness can also contribute significantly to relative responses because of varying effect of soil-structure interaction [1]. In the case of bridge structures the spatial variation of the ground motions and different interaction between bridge structures and subsoil can be expected, since large bridge dimension makes non-uniform soil along the bridge structure likely. In most of the bridge structures with conventional expansion joint the gap between adjacent girders is often small to ensure the bridge serviceability. In the case of strong earthquakes only adjusting the fundamental vibration frequencies of adjacent bridge structures is usually not sufficient to avoid poundings between bridge girders as observed in many major earthquakes in the past. Poundings inevitably damage bridge decks; they can also unseat the bridge span and cause its collapse.

In this study a new design philosophy is introduced. Unlike current design of bridges by accepting pounding between their girders in the event of strong earthquakes and by dealing with mitigating measures to reduce the pounding effect, the new design approach does not tolerate pounding by allowing large closing relative movement between adjacent bridge girders. This is achieved by using MEJ. With a MEJ it is possible to provide a large total gap between two girders when a number of small gaps is installed. Up to now MEJ is used to cope with large movements of long bridges due to thermal expansion and contraction. It can be used in any bridges in seismic design to provide gap size between girders to avoid pounding. Figure 4 shows two adjacent bridge segments with a MEJ in between. The longitudinal cross section of the MEJ is displayed in the upper part of the figure. Both ends of left and right girders are joined by an edge beam and several middle beams. Free and movable rubber is used to seal the gaps between the beams to ensure watertightness. The traffic loading is transferred from the joint to the adjoined bridge girders by rubber bearings and support beam. The bearings ensure a uniform movement of the beams. Details of MEJ can be found e.g. in [34].

The authors propose the usage of MEJ to cope with large earthquake-induced relative movement between bridge girders. When the number of the intermediate gaps is sufficient, they can provide the necessary clearance for relative movements of adjacent girders, and pounding will not take place. Consequently, mitigation measures and their subsequent maintenance become superfluous. Previous investigations on MEJ mainly focused on long-term fatigue behaviour of MEJ due to very large number of repeated vehicle loading and continuous opening and closing movements of the MEJ gaps. Other studies concentrated on the reduction of noise induced by traffic [35]. Studies of the suitability of MEJ to prevent pounding between bridge girders under strong earthquakes is limited [24], as discussed above. In this study the most significant design parameters of MEJ to

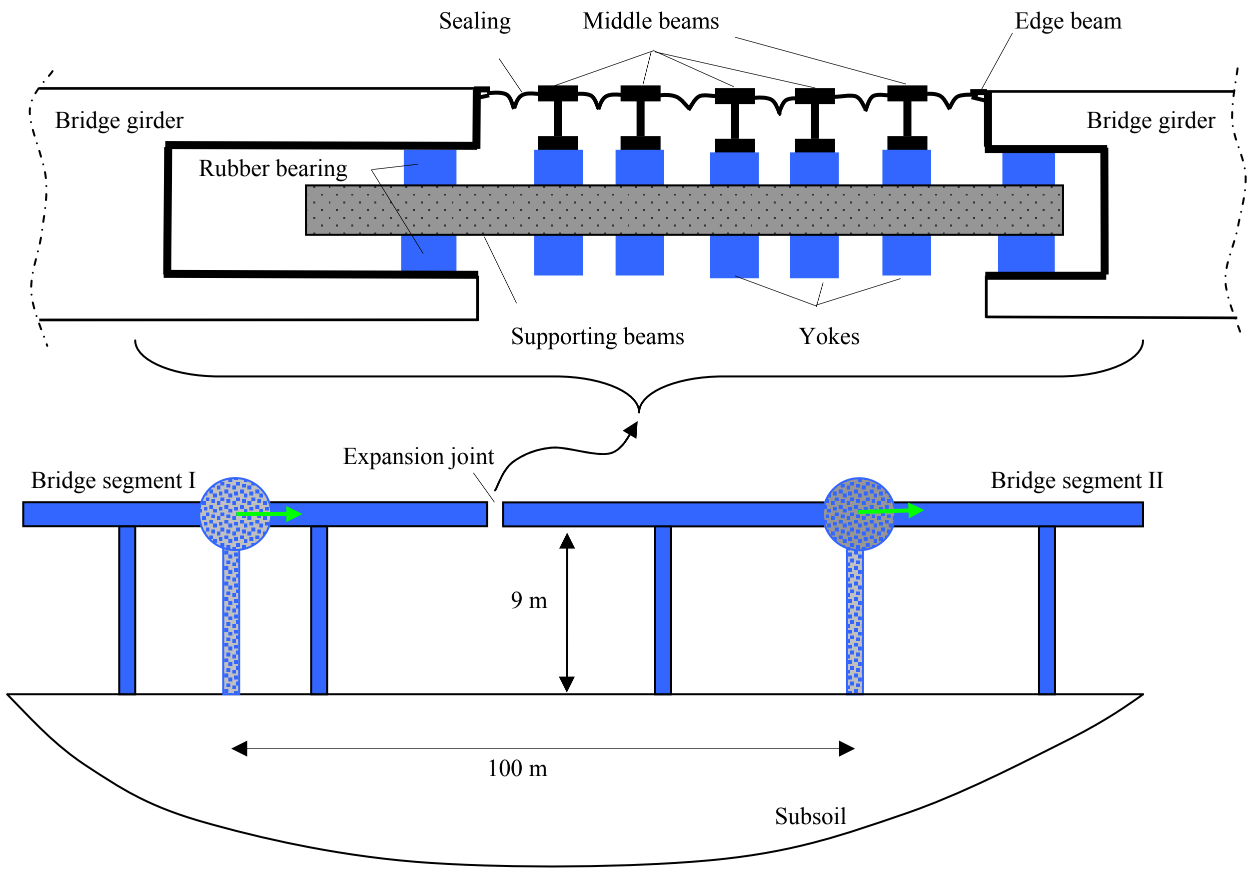


***Figure 4: Adjacent bridge segments with subsoil and modular expansion joint.***

cope with large girder movements without causing pounding due to strong earthquakes designed in the Japanese design code are investigated. The considered parameters are: The minimum total gap required to prevent pounding and the minimum opening movability required to prevent overstretching the rubber sealing.

## NECESSARY MINIMUM CLOSING RELATIVE MOVEMENTS

### Influence of spatial variation of ground excitations

Spatially varying ground excitations cause not only dynamic but also quasi-static responses. When the fundamental frequency of a structure is high relative to the dominant frequencies of the ground motions, the quasi-static response will govern the response of the structure. When the structure is relatively flexible, dynamic response will determine the structural response [36].

Figure 5 shows the influence of spatially varying ground motions on the mean values of the minimum total gap $u_c$ required to prevent pounding between the bridge girders. The mean values are the ensemble means of twenty time history analyses using twenty sets of independently simulated intermediately correlated ground motions as input. These twenty sets of ground motions are simulated to be compatible with the design spectrum of soft soil conditions, coherency loss function with intermediate correlation assumption and an apparent wave velocity $c_a$ of 500 m/s. It is assumed that the bridge structures are fixed at their base. To complete the investigation, uniform ground motion assumption and spatially varying ground motions with a time delay only are also considered. In uniform ground motion assumption, the simulated ground motion at the left support is used. In the case of ground motions with only a time delay, it is assumed that the ground motions at both supports are the same, again the time history at the left support is used, but that at the right bridge support occurs 0.2 s later than those at the left bridge segment because the separation distance between the two supports is 100 m. This simplification has been popularly used by researchers in modelling the ground motion spatial variations. When both adjacent structures have the same fundamental period ($T_1$ = $T_2$) an assumption of uniform ground excitation will not generate any relative response between two structures, i.e. no gap is required to avoid pounding. These results correspond well with the recommendation of current design regulations. A consideration of spatially varying ground excitations produces, however, not only in the case of $T_1$ = $T_2$ but in all considered periods a much larger required total gap, especially when the adjacent structure is relatively stiff (Figures 5(b) and (c)). Where equal fundamental period ($T_1$ = $T_2$) supposes to prevent the structures from pounding, a large total gap is actually needed because non-uniform ground motions also generate relative responses between the two structures. An assumption of a time delay due to wave propagation between the considered structural supports produces indeed more realistic results. However, it still underestimates the total gap required. The results reveal that not only a time delay but also the coherency loss in the ground motions are significant to obtain a realistic total gap required to avoid pounding. The recommendation of current design regulations to have structures with the same or similar fundamental periods can be applied, when both adjacent structures are relatively flexible. Even though this recommendation alone will not provide sufficient expansion gap between the bridge girders, the minimum total gap required to prevent pounding can be significantly reduced because the responses are governed by dynamic responses. When one of the structures or both of them are relatively stiff, this recommendation does not necessarily produce the smallest minimum total gap required.

### Influence of the frequency content of ground motions

Figure 6 shows the mean values of the minimum total gap required to avoid bridge girder pounding for soft, medium and hard soil conditions. It is assumed that the spatially varying ground motions are intermediately correlated with a wave apparent velocity $c_a$ of 500 m/s, and the bridge structures are fixed at their base. Although all ground accelerations have almost the same peak ground acceleration (PGA) of 6 m/s$^2$, the results are not the same. The minimum total gap required reduces with increasing stiffness of the local site, except in the case of $T_1$ = 1 s and $T_2$ between 1 s and 1.3 s, where the medium soil condition results in comparable relative responses to the hard soil condition.

The recommendation of equal or similar fundamental periods of adjacent structures reduces the minimum total gap only when the both bridge segments are relatively flexible (Figure 6(a)). The results show that PGA alone is not an adequate design parameter. The frequency contents and spatial variation of ground motion also significantly affect the relative responses. The results also show that both the period ratio and the absolute periods of the bridge segments strongly influence the total minimum gap required to prevent pounding.

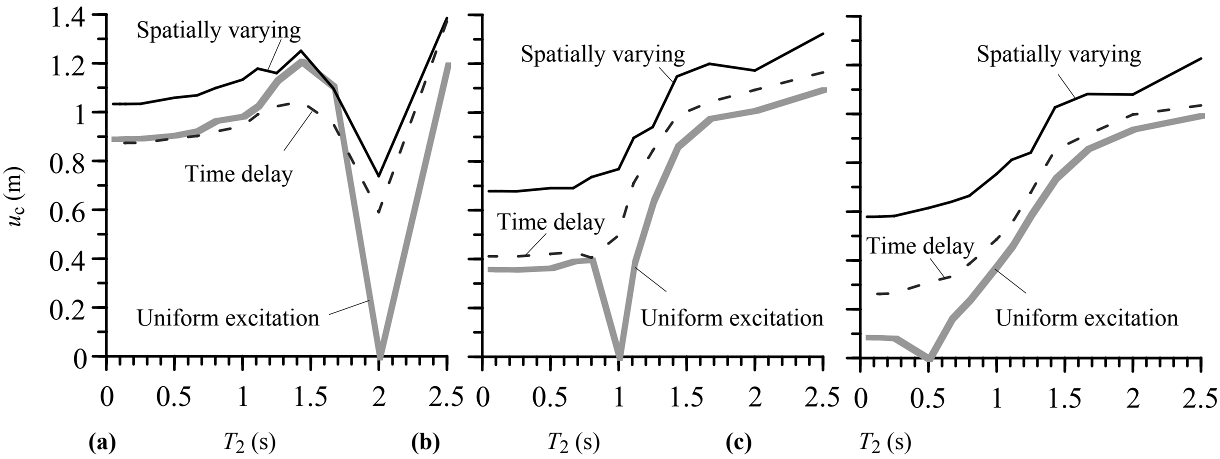


***Figure 5(a)-(c): Influence of spatial variation of ground excitations on the required minimum gap $u_c$ for (a) $T_1$ = 2 s, (b) $T_1$ = 1 s and (c) $T_1$ = 0.5 s.***

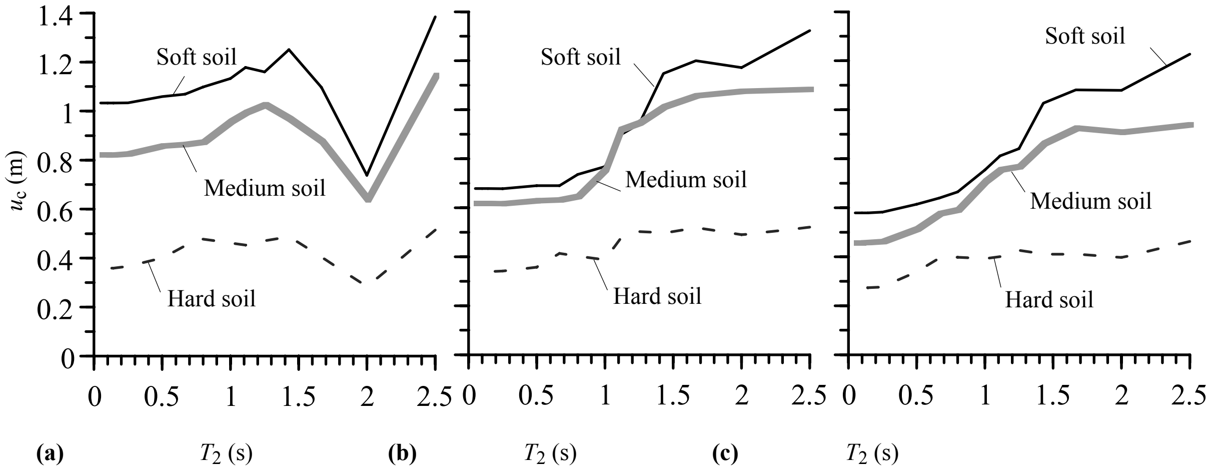


***Figure 6(a)-(c): Influence of soil type on the required minimum gap $u_c$ for (a) $T_1$ = 2 s, (b) $T_1$ = 1 s and (c) $T_1$ = 0.5 s.***

**Influence of coherency loss of ground motions**

Figure 7 displays the mean values of the minimum total gap required when the spatially varying ground motions are weakly, intermediately and highly correlated and are compatible to design spectrum of soft soil conditions. Fixed-base bridge segments and a wave apparent velocity $c_a$ of 500 m/s are assumed. With growing coherency loss -as expected- the minimum total gap increases. Only in the case of very flexible bridge segments ($T_1$ = 2 s and $T_2$ larger than 2 s) the highly and intermediately correlated ground excitations (bold solid and dash lines) produce comparable total gap required. This is expected because the effect of coherency loss between spatial ground motions is more significant to quasi-static response and is less pronounced to dynamic response. When the structures are relatively flexible, the dynamic response is dominant and therefore coherency loss effect is less prominent. The coherency loss of the ground motion spatial variation effect will have a significant contribution to the relative response of the adjacent structures and consequently to the minimum total gap when structures are relatively stiff.

Although the recommendation of equal fundamental periods may result in the smallest minimum total gap, however, it is not equal to zero. The results confirm the significance of the spatially varying ground motions in producing relative responses of adjacent structures.

**Influence of soil-structure interaction and spatial variation of ground motions**

Figure 8 shows the influence of soil-structure interaction (SSI) and the simultaneous effect of SSI and spatial variation of the ground motions on the mean values of the minimum total gap required. The considered input ground motions are compatible to design spectrum with soft soil conditions, intermediate cross correlation and a wave apparent velocity $c_a$ of 500 m/s. In the case of equal fundamental periods an assumption of uniform ground excitation will confirm that no gap is necessary. Even though SSI does exist, it has no contribution, since it is assumed that both bridge segments have the same slenderness and soil properties at both supports and thus experience the same soil-structure interaction. In the case of very flexible structures ($T_1$ = 2 s and $T_2$ is larger than 1.65 s in Figure 8(a)) SSI has also negligible contribution, when spatially varying ground motions are considered. The reason is that soft soil and flexible structures have negligible interaction effect. In this period range of $T_2$ above 1.65, SSI effect is insignificant.

The interaction between bridge structures and subsoil is pronounced, when the structures are stiff and soil is soft as shown in the results in lower period range of $T_2$ below 1.65 in Figure 8(a) or in all results in Figures 8(b) and 8(c). SSI results in a larger minimum total gap, and the simultaneous effect of SSI and spatial variation of the ground motions produce the largest minimum total gap required to avoid pounding.

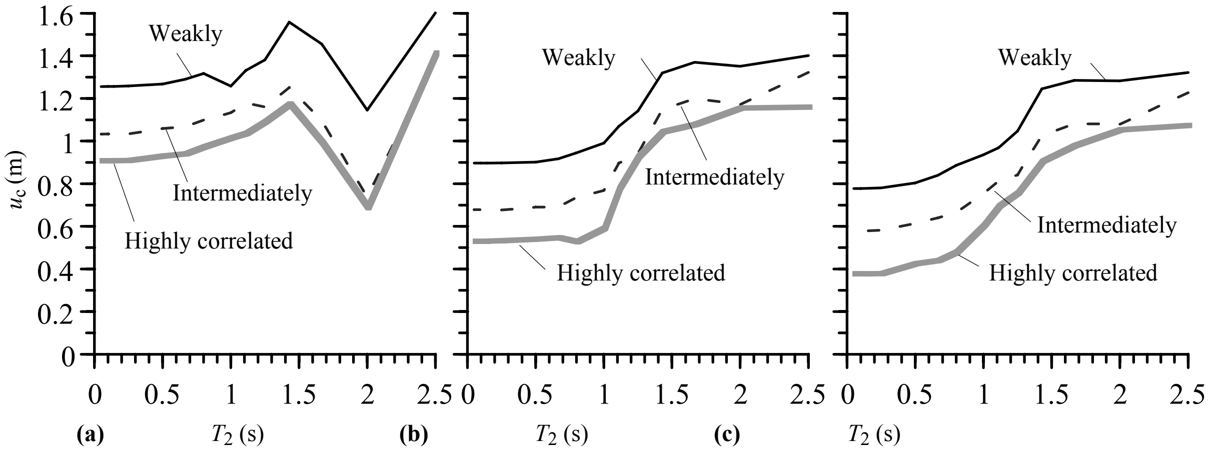


***Figure 7(a)-(c): Influence of coherency loss on the required minimum gap $u_c$ for (a) $T_1$ = 2 s, (b) $T_1$ = 1 s and (c) $T_1$ = 0.5 s.***

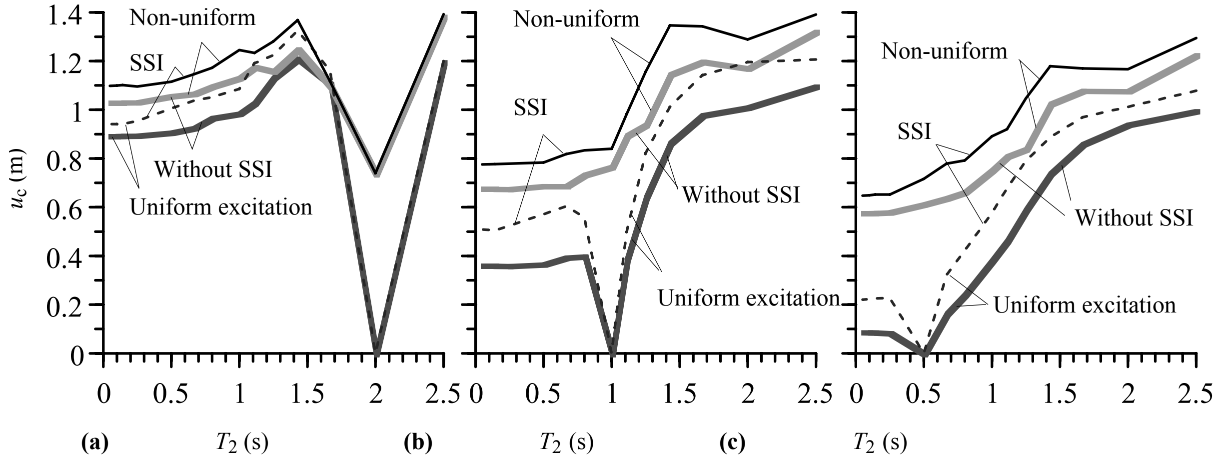


***Figure 8(a)-(c): Influence of SSI and spatial variation of ground motions on the required minimum gap $u_c$ for (a) $T_1$ = 2 s, (b) $T_1$ = 1 s and (c) $T_1$ = 0.5 s.***

## REQUIRED MINIMUM OPENING RELATIVE MOVEMENTS

**Influence of spatial variation of ground excitations**

MEJ fulfils its function when the total allowed relative movement is larger than the minimum total gap required to avoid pounding. Pounding will then not take place. Since under earthquake loading both closing and opening relative movements between the girders will occur, MEJ must also be able to cope with the largest possible opening movement without overstretching the rubber sealing between the MEJ beams during their opening movements.

Figure 9 shows the effect of spatial variation of the ground motions on the mean values of the minimum opening displacement $u_o$ of the expansion joint. This opening displacement should be used in the MEJ design to prevent damage to the rubber sealing. The results are obtained without considering SSI effect. The spatial ground motion inputs used are compatible to the design spectrum of soft soil conditions, with intermediate correlation and a wave apparent velocity $c_a$ of 500 m/s. For comparison, results corresponding to uniform ground motion and spatial ground motions with a 0.2 s time delay only as discussed above are also calculated and displayed in the figure.

Similar observation can be made as that of the minimum total gap necessary to prevent bridge girder from pounding. If the bridge structures are flexible (Figure 9(a)) the smallest minimum opening relative displacement occurs when the recommendation of current design regulations by adjusting the vibration periods of adjacent structures close to each other is followed. A consideration of a time delay only will underestimate the required minimum opening displacement. This approach may even produce smaller results than those caused by uniform ground excitation ($T_1$ = 2 s and $T_2$ below 1.7 s). The considered cases show that if one of the bridge segments is relatively stiff with a fundamental frequency higher than or equal to 1 Hz, the code recommendation will not provide the smallest minimum opening displacement necessary to prevent damage to the covering seals. The recommendation of current design regulations will clearly underestimate the necessary minimum opening displacement of MEJ.

**Influence of the frequency content of ground motions**

Figure 10 shows the mean values of the minimum opening displacement obtained from 20 time history analyses with ground motions simulated according to design spectra for different soil conditions. It is assumed that all ground motions are intermediately correlated with a wave apparent velocity $c_a$ of 500 m/s, and the bridge structures are fixed at their base without soil-structure interaction. Similar observation can be made as the case of minimum total gap necessary to prevent bridge girder from pounding. In general, the opening displacement increases with lower dominant frequencies of the ground motions, because lower frequency content means larger ground displacements. Consequently, larger spatially varying ground displacements will then result in larger quasi-static responses.

When both bridge structures have the same fundamental period and are relatively flexible, the smallest required opening movability of a MEJ will be achieved (Figure 10(a)). This is no longer the case, when one of the bridge structures is relatively stiff (Figures 10(b) and 10(c)). The results reveal the significant influence of the ground motion frequency contents. Soft soil ground motions can cause more than 100 % larger desired minimum opening movabilty than hard soil ground motions.

**Influence of coherency loss of ground motions**

Figure 11 displays the mean values of the minimum opening movability of a MEJ due to weakly, intermediately and highly correlated spatially varying ground motions. It is assumed that the bridge structures are fixed at their base, and the wave apparent velocity $c_a$ of the soft soil ground motions is 500 m/s. Although similar observation of the results can be made as in the case of minimum total required gap to avoid pounding, if both bridge structures are relatively flexible the minimum opening movability increases with the loss of the coherency in all considered fundamental period ratios. A comparison of the results reveals that when both bridge structures have the same fundamental period the stiffer the structures are, the smaller the desired minimum opening movability will be. However, more investigations are necessary to find the optimum design requirement.

**Influence of soil-structure interaction and spatial variation of ground motions**

Figure 12 shows the influence of SSI and spatial variation of the ground motions on the mean values of the minimum opening movability required to prevent damage of the MEJ sealing. It is assumed that the spatially varying ground motions are intermediately correlated with a wave apparent velocity $c_a$ of 500 m/s and are compatible to design spectrum for soft soil conditions.

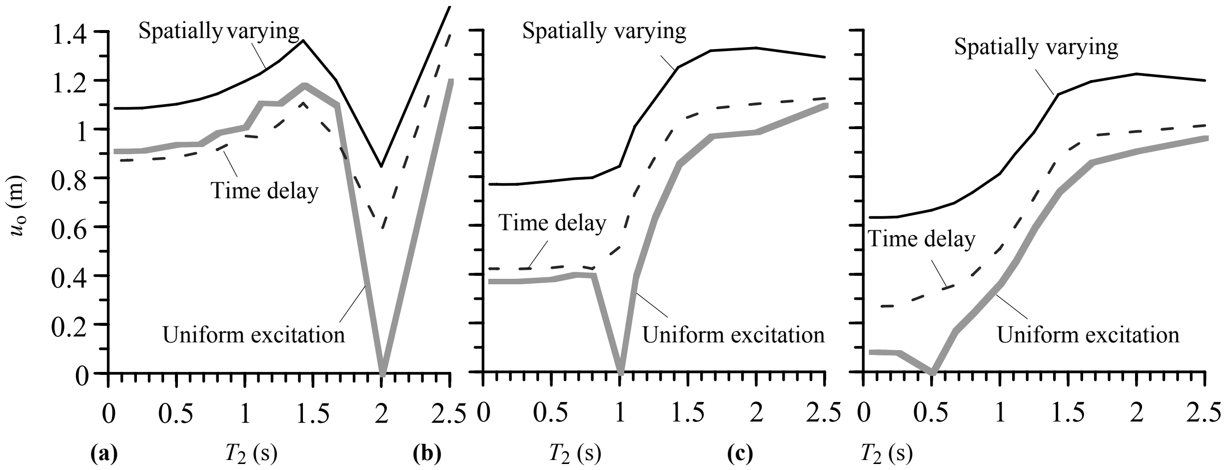


***Figure 9(a)-(c): Influence of spatial variation of ground excitations on the required minimum opening movability $u_o$ for (a) $T_1$ = 2 s, (b) $T_1$ = 1 s and (c) $T_1$ = 0.5 s.***

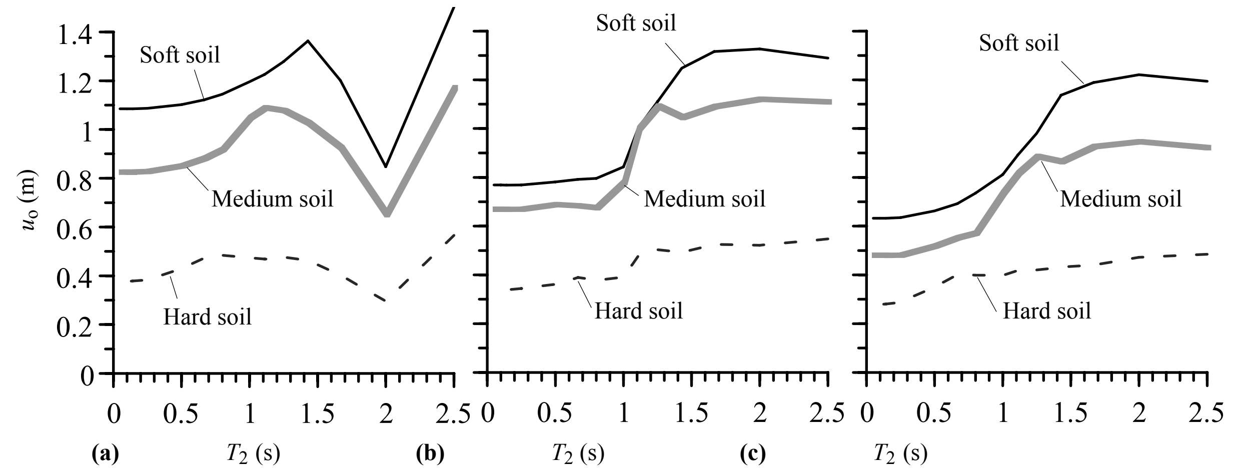


***Figure 10(a)-(c): Influence of soil type on the required minimum opening movability $u_o$ for (a) $T_1$ = 2 s, (b) $T_1$ = 1 s and (c) $T_1$ = 0.5 s.***

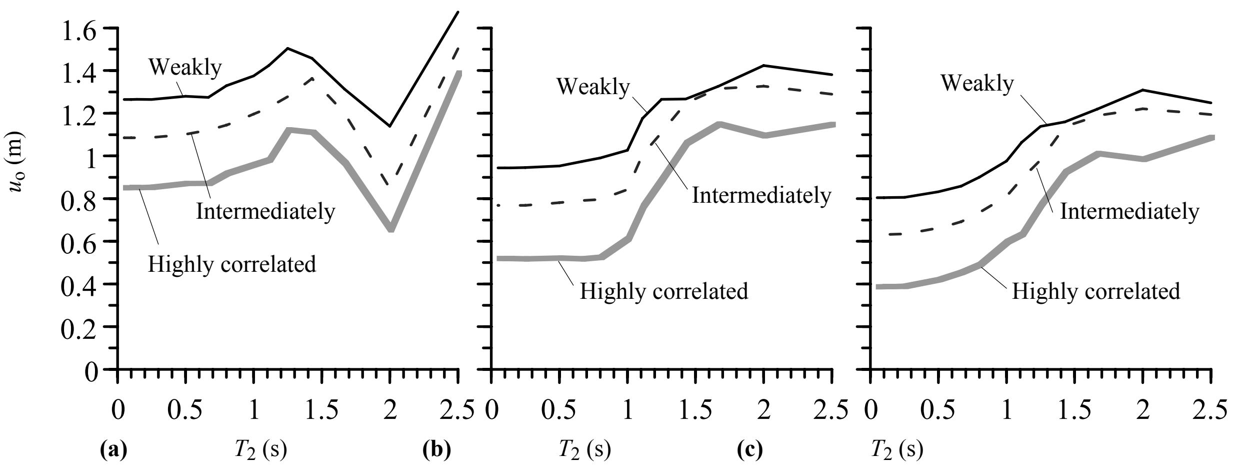


***Figure 11(a)-(c): Influence of coherency loss on the required minimum opening movability $u_o$ for (a) $T_1$ = 2 s, (b) $T_1$ = 1 s and (c) $T_1$ = 0.5 s.***

When one of the bridge structures is relatively stiff, SSI always increases the desired opening movability. When both bridge structures are relatively flexible, SSI has minimum effect (Figure 12(a) for $T_2$ larger than 1.65 s). The explanation is given in previous section regarding the minimum total gap required to prevent girder pounding in Figure 8(a). The results here also show that the largest movability is required when spatially varying ground motions and SSI are considered. In these considered cases the effect of spatial variation of the ground motions is more dominant than that of SSI.

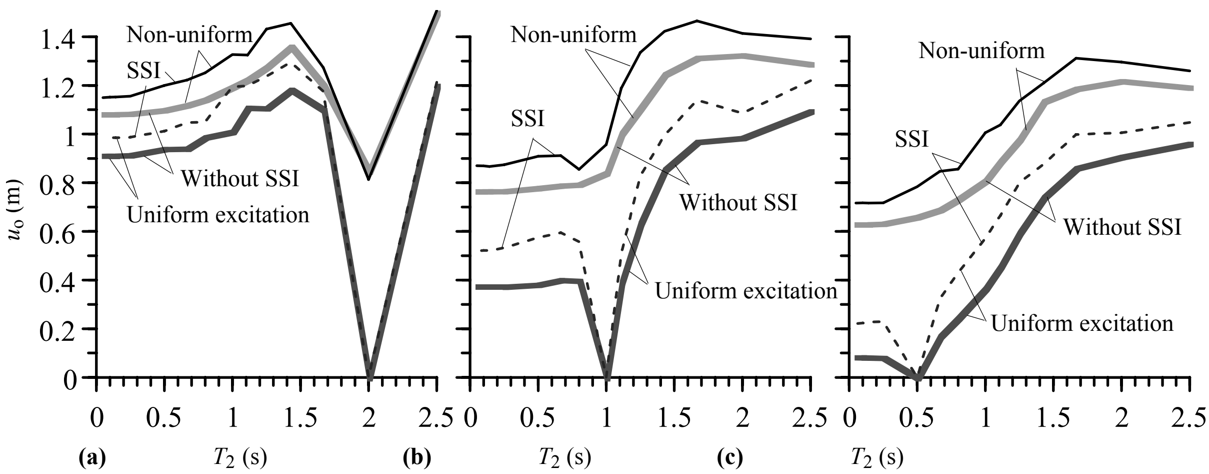


***Figure 12(a)-(c): Influence of SSI and spatial variation of ground motions on the required minimum opening movability $u_o$ for (a) $T_1$ = 2 s, (b) $T_1$ = 1 s and (c) $T_1$ = 0.5 s.***

## CONCLUSIONS

In this work a new design philosophy is introduced. Unlike current conventional expansion joints used in bridges with a gap of only a few centimetres that are usually insufficient to completely preclude poundings between bridge girders during strong earthquake excitations, the new design approach proposes the usage of modular expansion joints (MEJ) which allow for large total relative displacement in the joints to completely prevent pounding between bridge girders. Modular expansion joints enable a large total gap by having a number of intermediate gaps. Since each of these gaps remains small, the serviceability of the bridge is ensured. Using MEJs to resist earthquake loadings, it must be designed with sufficient closing and opening movements to prevent the adjacent bridge girders from pounding or overstretching the rubber sealing of the intermediate gaps.

The necessary closing and opening displacements of a MEJ corresponding to the design response spectra specified in the Japanese seismic design code are estimated in this study. In total 100 sets of spatially varying ground motions are used. The numerical analysis addressed the influence of soil-structure interaction and spatial variation of ground motions on the minimum total gap required to avoid pounding between adjacent bridge girders and the minimum opening movability of MEJ required to prevent damages to the rubber sealing.

The investigations reveal:

The recommendation of current design regulations to adjust the fundamental periods of the adjacent structures does produce the smallest minimum total gap and the smallest minimum opening movability when both adjacent structures are relatively flexible.

A consideration of the ratio of the fundamental periods of the adjacent bridge structures is insufficient. The absolute vibration periods also strongly affect the response.

An assumption of a time delay owing to seismic wave propagation is not sufficient to obtain a realistic design value. This assumption usually underestimates the required closing and opening displacement of a MEJ.

Spatially varying ground motions corresponding to soft soil conditions are associated with larger relative ground displacements than those of hard soil conditions, which produce larger relative responses of adjacent bridge structures, therefore larger required closing and opening displacement of a MEJ.

The simultaneous effect of SSI and spatial variation of ground motions produces the largest desired movement in a MEJ.

## REFERENCES


1 Chouw, N., Hao, H. (2008) “Significance of SSI and non-uniform near-fault ground motions in bridge response I: Effect on response with conventional expansion joint”. *Engineering Structures*, **30**: 141-153.

2 Moehle, J., Fenves, G., Mayes, R., Priestley, N., Seible, F., Uang, C.M. (1995) “Highway bridges and traffic management”. *Earthquake Spectra*, **11**(S2): 287-372.

3 Park, R., Billings, I.J., Clifton, G.C., Cousins, J., Filiatrault, A., Jennings, D.N., Jones L.C.P., Perrin, N.D., Rooney, S.L., Sinclair, J., Spurr, D.D., Tanaka, H., Walker, G. (1995) “The Hyogo-ken Nanbu Earthquake of 17 January 1995”. *Bulletin of the New Zealand Society for Earthquake Engineering*, **28**(1): 98.

4 Japan Society of Civil Engineers (JSCE) (1999) “*The 1999 Ji-Ji earthquake, Taiwan –investigation into damage to civil engineering structure*”. In: Hamada, M., Nakamura, S., Ohsumi, T., Megro, I., Wang, E. (eds.), 160.

5 Elnashai, A.S., Kim, S.J., Yun, G.J., Sidarta, D. (2007) “*The Yogyakarta earthquake on May 27, 2006*”. Mid-America Earthquake Center, Report No. 07-02, 57.

6 Anagnostopoulos, S.A. (1988) “Pounding of buildings in series during earthquakes”. *Earthquake Engineering and Structural Dynamics*, **16**: 443-456.

7 Anagnostopoulos, S.A., Sliliopoulos, K.V. (1992) “An investigation of earthquake induced pounding between adjacent buildings”. *Earthquake Engineering and Structural Dynamics*, **21**: 289-302.

8 Athanassiadou, C., Penelis, G.G., Kappos, A.J. (1994) “Seismic response of adjacent buildings with similar or different dynamic characteristics”. *Earthquake Spectra*, **10**(2): 293-31.

9 Penzien, J. (1997) “Evaluation of building separation

distance required to prevent pounding during strong earthquakes". *Earthquake Engineering and Structural Dynamics*, **26**: 849-859.

10 Hao, H., Liu, X. (1998) "Estimation of required separations between adjacent structures under spatial ground motions". *Journal of Earthquake Engineering*, **2**(2): 197-216.

11 Hao, H., Shen, J. (2001) "Estimation of relative displacement of two adjacent asymmetric structures". *Earthquake Engineering and Structural Dynamics*, **30**: 81-96

12 Luco, J.E., Barros, F.C.P. (1998) "Optimal damping between two adjacent elastic structures". *Earthquake Engineering and Structural Dynamics*, **27**: 649-659.

13 Hao, H., Zhang, S. (1999) "Spatial ground motion effect on relative displacement of adjacent building structures". *Earthquake Engineering and Structural Dynamics*, **28**: 333-334.

14 Rahman, A.M., Carr, A.J., Moss, P.J. (2001) "Seismic pounding of a case of adjacent multiple-storey buildings of differing total heights considering soil flexibility effects". *Bulletin of the New Zealand Society for Earthquake Engineering*, **34**(1): 40-59.

15 Chouw, N. (2002) "Influence of soil-structure interaction on pounding response of adjacent buildings due to near-source earthquakes". *JSCE Journal of Applied Mechanics*, **5**: 543-553.

16 DesRoches, R., Muthumar, S. (2002) "Effect of pounding and restrainers on seismic response of multiple-frame bridge". *ASCE Journal of Structural Engineering*, **128**: 860-869.

17 Ruangrassamee, A., Kawashima, K. (2003) "Control of nonlinear bridge response with pounding effect by variable dampers". *Engineering Structures*, **25**: 593-606.

18 Zhu, P., Abe, M., Fujino, Y. (2004) "Evaluation of pounding countermeasures and serviceability of elevated bridges during seismic excitation using 3D modelling". *Earthquake Engineering and Structural Dynamics*, **33**: 591-609.

19 Yashinsky, M. and Karshenas, M.J. (2003) "*Fundamentals of seismic protection for bridges*". Earthquake Engineering Research Institute, Monograph MNO-9, 184.

20 Hao, H. (1998) "A parametric study of the required seating length for bridge decks during earthquakes". *Earthquake Engineering and Structural Dynamics*, **27**: 91-103.

21 Zanardo, G., Hao, H., Modena, C. (2002) "Seismic response of multi-span simply supported bridges to a spatially varying earthquake ground motion". *Earthquake Engineering and Structural Dynamics*, **31**: 1325-1345.

22 Zhu, P., Abe, M., Fujino, Y. (2002) "Modelling three-dimensional non-linear seismic performance of elevated bridges with emphasis on pounding of girders". *Earthquake Engineering and Structural Dynamics*, **31**: 1891-1913.

23 Tongaonkar N. P., Jangid, R.S. (2003) "Seismic response of isolated bridges with soil-structure interaction". *Soil Dynamics and Earthquake Engineering*, **23**: 287-302.

24 Chouw, N., Hao, H. (2008) "Significance of SSI and non-uniform near-fault ground motions in bridge response II: Effect on response with modular expansion joint". *Engineering Structures*, **30**: 154-162.

25 Ambraseys, N.N., Douglas, J. (2003) "Near-field horizontal and vertical earthquake ground motions". *Soil Dynamics and Earthquake Engineering*, **23**: 1-18.

26 Japan Society of Civil Engineers (JSCE) (2000.) "*Earthquake resistant design codes in Japan*". Maruzen, Tokyo

27 Kodama, T., Chouw, N. (2002) "Soil-structure interaction with uplift due to near-source earthquakes". *Proceedings of European conference on structural dynamics*, Rotterdam, Balkema, **2**: 1375-1380.

28 Chouw, N. (1994) "*Analysis of structural vibration considering the dynamic transmitting behaviour of soil*". Technical report 94-3, Ruhr University Bochum (in German)

29 Hao, H. (1989) "*Effects of spatial variation of ground motions on large multiple-supported structures*". *Report no. UCB-EERC 89/06*, UC Berkeley

30 Hao, H., Oliveira CS., Penzien, J. (1989) "Multiple-station ground motion processing and simulation based on SMART-1 array data". *Journal of Nuclear Engineering Design*, **111**: 293-310.

31 American Association of State Highway and Transportation (AASHTO) (1998) "*Load and resistance factor design specifications for highway bridges*". Washington DC

32 CALTRANS seismic design criteria (1999) "*Design manual –version 1.1*". California Department of Transportation, Sacramento, California

33 Japan Road Association (JRA) (2004) "*Specifications for highway bridges –Part V seismic design*". 5$^{th}$ ed., 406 (in Japanese)

34 Dexter, R.J., Mutziger, M.J., Osberg, C.B. (2002) "*Performance testing for modular bridge joint systems*". National cooperative highway research program, Report 467, 73

35 Ravshanovich, K.A., Yamaguchi, H., Matsumoto, Y., Tomida, N., Uno, S. (2007) "Mechanism of noise generation from a modular expansion joint under vehicle passage". *Engineering Structures*, **29**: 2206-2218.

36 Hao, H. (1993) "Arch response to correlated multiple excitations". *Earthquake Engineering and Structural Dynamics*, **22**: 389-404.