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Seismic Design of Bridges for Prevention of Girder Pounding

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ABSTRACT: In design of conventional bridges the gap between bridge spans is usually only a few centimetres. For such an expansion joint poundings between adjacent bridge girders during strong earthquake shaking are usually unavoidable. Pounding often causes damages to girders. In extreme situations it may push one of the bridge decks off the support. In this work a new design philosophy using a modular expansion joint (MEJ) is introduced. So far MEJs have been used mainly to cope with large thermal expansion and contraction of long bridges. For proper design of bridges to avoid the consequence of poundings under strong earthquakes not only a minimum total gap but also the maximum opening movement of the joint are essential. In this study the simultaneous effect of varying vibration properties of adjacent bridge spans, spatially varying ground excitations and soil-structure interaction on the total closing and opening movements of a MEJ, required to eliminate possible pounding and to ensure the join in perfect working order, is estimated, and the main influence factors are discussed.

Keywords: Modular expansion joint, spatial variation, soil-structure interaction, separation distance, pounding prevention

1 INTRODUCTION

Pounding can cause substantial damage to neighbouring structures when they are sufficiently close to each other. Despite research effort in the past decades and recommendations provided in almost all seismic design codes, pounding damage of bridge girders has still been observed in many recent major earthquakes, e.g. during the 1994 Northridge earthquake (Moehle *et al.* 1995), the Kobe earthquake (Park *et al.* 1995), the Chi-Chi earthquake (JSCE 1999) and the Yogyakarta earthquake (Elnashai *et al.* 2007). Current design regulations, e.g. CALTRANS (1999), AASHTO (1998) and JRA (2004), recommend that adjacent structures should have a sufficient separation distance and the same or at least similar fundamental vibration period to ensure their in-phase vibrations. The idea is that this in-phase overall vibration will then prevent the adjacent structures from colliding. This recommendation, however, is made under the assumption that the structures experience the same ground excitation, and their behaviour is determined only by the structural properties. In the case of adjacent buildings an assumption of same ground excitation is justifiable. However, their dynamic behaviour can be affected by the different footing properties and non-uniform ground conditions as well as by the interaction between buildings and subsoil, which may induce out-of-phase responses between adjacent buildings. In the case of bridge structures, besides soil-structure interaction (SSI), inevitable spatially non-uniform ground excitation at the neighbouring bridge pier supports is another factor that may produce out-of-phase responses of adjacent bridge spans. In such a case the current design recommendations can cause an adverse effect (Chouw & Hao, 2008). While a minimum distance between buildings is a possible measure for avoiding pounding damage, in the case of bridge structures a large gap between adjacent girders will strongly hinder the passage of traffic. An adjustment of the fundamental frequencies of the adjacent bridge structures may not be a sufficient or suitable approach to reduce out-of-phase responses, because bridge structures will more likely experience spatially non-uniform ground excitation. To overcome this difficulty, in this work a new design philosophy is introduced.

2 CAUSE OF RELATIVE MOVEMENT BETWEEN BRIDGE GIRDERS

There are a number of sources that induce relative movement between bridge girders. They include:

- Unequal vibration properties of adjacent bridge structures
- Spatial variation of ground motions
- Different soil-structure interaction because of varying soil properties at bridge supports and because of different footings and slenderness ratios of adjacent bridge piers

The response of a structure is as good as the assumption of the excitation. In the case of earthquake loading the ground excitation of the structure is determined not only by the geological properties along the path of wave propagation from the source to the structural site and the properties of the local site, but also by the stiffness ratio between the structural footing and the supporting ground. The embedment of the footing and the interface between footing and subsoil can also alter the characteristics of the incoming waves and consequently the ground excitation of the structure.

The inertial forces activated by the excitation cause vibrations of the whole structure. If the structure is low, the structural footing tends to move laterally from side to side. This movement initiates pressure waves at both sides of the footing and at the same time shear waves from the bottom interface of the footing. If the structure is tall, the footing rocks because of the bending movement of the whole structure. This footing movement activates mainly pressure waves at both edges of the footing and shear waves at both sides of the footing. Since vibrating footing is a wave source, different movement of the footing means different wave propagation, and consequently different dynamic soil stiffness and radiation damping will be initiated.

In the case of long-extended structures, like bridges, the soil plays another significant role. Since seismic waves need time to travel from one bridge pier footing to another pier support, the ground excitation at the adjacent support experiences a time delay. Additionally, the soil properties between bridge supports are heterogeneous and non-uniform; therefore ground motions at the supports are incoherent. Consequently, the response of structures to spatially varying ground excitations is not the same as that to commonly-assumed uniform ground motions. In the case of bridge structures the spatial variation of the ground excitations causes different relative response between the adjoining bridge segments as those under the assumption that both bridge segments experience the same ground motions.

When adjacent bridge segments have different slenderness, even if they have the same fixed-base fundamental frequency and experience the same ground excitation and the subsoil at the bridge piers is uniform with the same properties, relative response between the bridge segments will occur. This is because both bridge structures will interact differently with the supporting soil. Consequently, each structure responds differently and contributes to their relative movements. The influence of the soil on the period $\tilde{T}_i$ of the system (bridge segment including subsoil) can be easily seen from Equation (1).

$$\tilde{T}_i = T_i \sqrt{1 + \frac{k_i}{k_{ix}} \left(1 + \frac{k_{ix}\, h_i^2}{k_{i\phi}}\right)} \qquad (1)$$

where T_i is the period of the bridge segment with an assumed fixed base (without the influence of supporting soil). k_i, k_{ix}, $k_{i\phi}$ are the bending stiffness of the bridge structure, and the static soil stiffness for horizontal and rocking movements of the assumed rigid bridge footing, respectively.

In Equation (1) the influence of the vertical soil stiffness is assumed to be negligible. In reality the soil stiffness is frequency dependent which makes the influence of subsoil even more complex (see e.g. Sieffert & Cevaer 1992). From this equation and Figure 1 it is obvious that even if both bridge segments have the same fixed-base fundamental period ($T_1 = T_2$) and the same supporting ground ($k_{1x} = k_{2x}$ and $k_{1\phi} = k_{2\phi}$), the different structural heights ($h_1 \neq h_2$) will cause different system periods, and consequently, each bridge segment will respond differently, even if the ground excitation is the same.

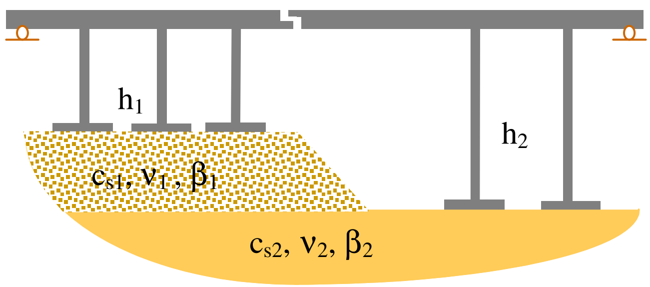

Figure 1 Soil-structure system.

In reality the local site at adjacent bridge pier supports is normally not the same. Hence, non-uniform soil properties contribute additionally to the relative response between the bridge structures.

Because of the nature of the structures to span a valley or river, often the subsoil is soft. The profile of soft supporting sediments can additionally amplify the incoming seismic waves and consequently the ground excitation of the bridge supports. Figure 1 shows a possible complex system. The adjacent bridge structures have different slenderness, and the supporting subsoil has different profiles and properties: shear wave velocity c_s, Poisson's ratio ν and material damping β. Several causes of structural poundings are described, e.g. in the reference (Chouw 2005).

The consequence of relative girder movements for adjacent bridge structures is damage to the bridge decks or even collapse. When closing relative movements are larger than the gap between adjacent girders, pounding between the girders will take place. Depending on the magnitude of the relative girder movement the damage can result in a bridge that is out of function.

When opening relative movements are larger than the seat length at the expansion joint unseating and a subsequent collapse of the bridge deck can happen. To prevent the consequence of large opening relative movements between bridge girders several measures have been developed and applied, e.g. stoppers and restrainers at the expansion joints (see e.g. Yashinsky & Karshenas 2003). It is, however, so far not possible to prevent girder pounding under strong earthquakes, because in order to ensure a serviceability of the bridge the gap at the expansion joints should be not more than a few centimetres. This small gap is, however, usually not sufficient to prevent adjacent bridge decks from colliding into each other. In this work a new design approach is introduced that enable large closing movement at the expansion joint without causing any pounding and at the same time the bridge serviceability is ensured.

3 NEW DESIGN PHILOSOPHY

Recently, to cope with large thermal expansion and contraction of long bridges more and more modular expansion joints (MEJ) are used. Figure 2 shows two segments of a bridge with a MEJ. The upper figure displays the cross section of the joint in the longitudinal direction of the bridge. The bridge segments are connected by edge beams at both girder ends and by middle beams. Support beams and rubber bearings transfer the traffic loading from the joint to the adjoining bridge girders. To ensure watertightness of the joint, free and moveable rubber sealing is installed between the beams. The bearings ensure that the beams move uniformly. Details of MEJ can be found, e.g. in the work by Dexter *et al.* (2002).

The authors propose to apply the ability of MEJ to mitigate the pounding problems due to large relative movement between bridge girders. Up to now, the suitability of MEJ to mitigate pounding damages of girders under strong earthquakes is unknown.

So far investigations of MEJ have been focused mainly on traffic-induced noise (e.g. Ravshanovich et al., 2007) and long-term MEJ fatigue behaviour due to repeated vehicle loading and continuous opening and closing movements of the MEJ beams.

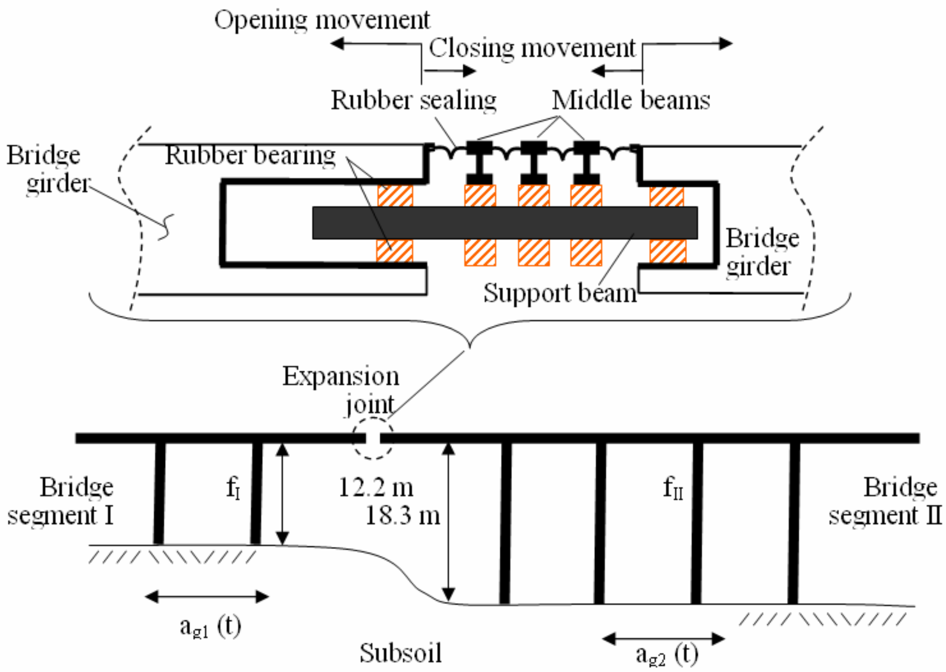

Figure 2 Bridge structures with subsoil and modular expansion joint.

The most significant requirement for proper design of a MEJ to cope with strong earthquake induced relative movement between bridge girders is the minimum total gap between MEJ beams to prevent pounding. To ensure a perfectly working condition the joint must also be able to open without tearing off the seals. Since the MEJ system ensures a uniform movement between the beams, in the investigation the influence of the rubber sealing is considered to be negligible. Instead, the investigation focuses on the most significant influence factors identified in previous studies (Chouw & Hao 2008):

- Characteristics of the spatially varying ground excitation: coherency loss and wave apparent velocities
- Ratio of the fundamental frequencies of the adjacent bridge structures
- Interaction between bridge structures and subsoil
- Combined effect of these factors

The considered left and right bridge structures in Figure 2 have the heights of 12.2 m and 18.3 m, respectively. To focus on the influence factors it is assumed that both structures have very similar fixed-base fundamental frequencies with a ratio f_{II}/f_I of 0.99. The soil is assumed to be a half space with a shear wave velocity c_s of 100 m/s, a density ρ of 2000 kg/m^3 and a Poisson's ratio ν of 0.33.

The bridge structures with their footings and the subsoil are described by finite elements and boundary elements, respectively.

Because in the 2D analysis an exact shape function (continuous-mass model) is used, only one finite element is necessary for each bridge pier and each girder. To couple the footing with the subsoil, 8 boundary elements are used to model the supporting soil at each footing-soil interface. The algorithm for the calculation of the girder responses with nonlinear soil-structure interaction is described in the reference (Chouw & Hao 2008).

For simplicity the multiple piers of the left and right bridge segments are described as a single pier, and the distance between these left and right modeled piers is assumed to be 100 m. It should be noted that to obtain a precise result the influence of multiple piers of each bridge segment and different ground excitations of these piers should be considered.

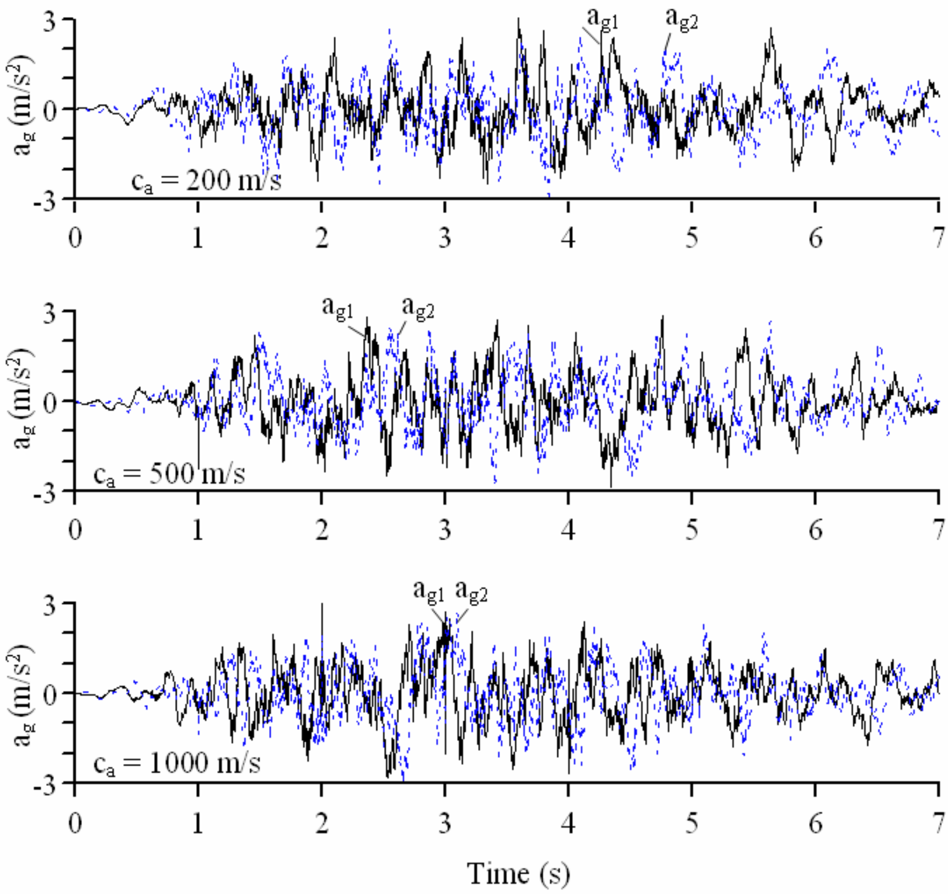

Figure 3 Simulated ground motions with different wave apparent velocities c_a.

Simplifying the multiple piers into one actually overestimates the ground motion spatial variation effect as multiple piers tend to average out such effect (Hao 1997). Although the simplified model is used, this study is a clear advancement compared to other studies, e.g. that performed by DesRoches & Mutkukumar (2002), where the influence of the pier, subsoil and spatial variation of the ground excitation is not considered at all. Figure 3 shows the influence of wave apparent velocity c_a on the spatial variation of the ground motions at the two distant bridge pier locations.

The ground motions are simulated based on a near-source ground motion model introduced by Ambraseys & Douglas (2003). The dominant frequencies of the simulated ground motions range between 2.5 Hz and 12.5 Hz with the peak ground acceleration of 3 m/s^2. The considered wave apparent velocities c_a are 200 m/s, 500 m/s and 1000 m/s. With increasing wave velocity the delay of the ground motions at the right bridge pier support decreases as the occurrence of the peak motions a_{g1} and a_{g2} shows. However, the spatial variation of the ground motions is not only characterized by the time delay but also by the coherency loss.

In this study three degrees of coherency loss are considered, which represent weakly, intermediately and highly correlated spatial ground motions at the two bridge pier supports.

For each parameter of the considered ground motion twenty sets of spatially non-uniform ground motions are simulated. In total 100 sets of ground motions are generated. Details of the ground motion simulation are given in (Chouw & Hao, 2008).

4 DESIGN PARAMETERS

4.1 Closing relative movements

Figure 4 displays the combined influence of SSI, the wave apparent velocity c_a and the coherency loss on the mean values of the minimum total MEJ gap required to avoid girder pounding. The total gap g_c is the sum of each gap between the MEJ beams (see Figure 2).

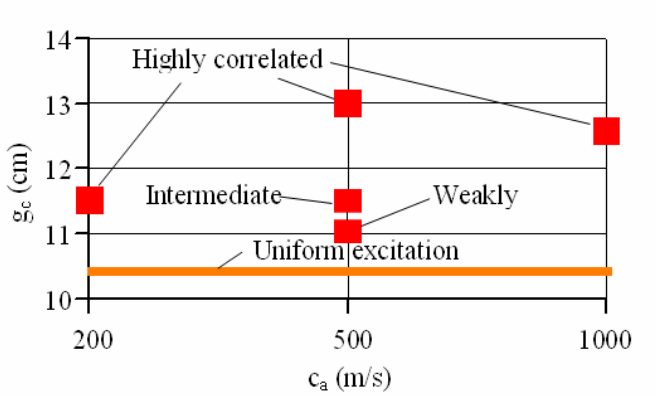

Figure 4 Minimum required total gap g of a MEJ.

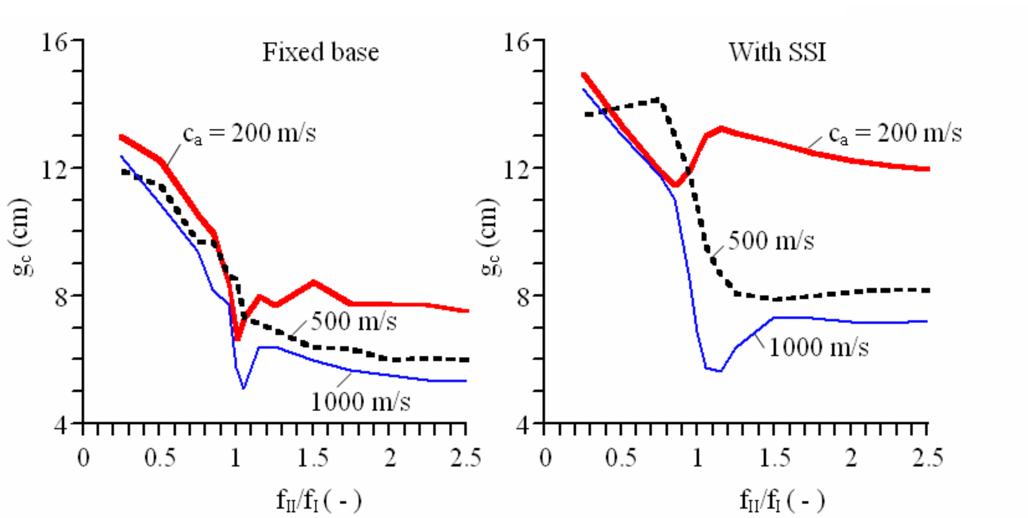

Figure 5 Dependence of the minimum gap g_c required on the frequency ratio f_{II}/f_I, the apparent wave velocity c_a of the spatially varying ground motions and SSI.

If fixed-base structures and uniform ground excitation are assumed, the minimum gap g_c is only 0.59 cm. This is to be expected, because both structures with assumed fixed base have very similar fundamental frequencies ($f_{II}/f_I = 0.99$).

If the effect of the subsoil is considered, the required gap is not similar as one might expect. Even though the frequencies of the two structures without considering SSI are similar and both structures experience the same ground excitation, owing to their different structural slenderness both bridge structures interact with their ground differently. The unequal SSI effect causes relative movements, and consequently a much larger minimum required total MEJ gap g_c of 10.42 cm. In Figure 4 it is indicated as a horizontal solid line.

The results show that an assumption of uniform ground excitation clearly underestimates the minimum required total gap of a MEJ to avoid pounding, especially when the structures are assumed to be fixed at their base. In the case of highly correlated spatially varying ground motions the minimum required gap does not decrease with higher wave apparent velocity c_a. In the case of the wave apparent velocity c_a of 500 m/s the minimum total gap also does not decrease with less coherency loss (e.g. the highly correlated case) as one would expect. These results reveal that the minimum total MEJ gap cannot be related to a single influence factor, because the combined influence of these factors is dominant.

Another influence factor, not considered so far, is the ratio f_{II}/f_I of the fixed-base fundamental frequencies of the adjacent bridge structures. The involvement of this factor in causing relative girder responses is displayed in Figure 5.

To enable a clear interpretation of the considered factors the effect of the structural slenderness is neglected. It is assumed that both adjacent structures have the same height of 9 m. Hence, the relative girder movement due to unequal interaction between the structures and soil is not considered. As a reference the case of structures with assumed fixed-base is displayed on the left side of Figure 5.

The results clearly show that the recommendation of current design regulations to avoid relative girder movement by designing structures with similar or equal fundamental frequencies is not adequate, when spatially non-uniform ground excitation does occur. In fact the fixed-base frequency ratio cannot be used as the only design parameter.

At the frequency ratio $f_{II}/f_I = 1.0$ the minimum total gap does not have the smallest value, and this value is definitely not equal to zero. In the investigation it is assumed that the spatially varying ground motions are highly correlated. In both cases, with and without SSI, the influence of the frequency ratio f_{II}/f_I is obvious.

In the higher ratio range, f_{II}/f_I above 1.15, the influence of the wave apparent velocity c_a is dominant. As expected, the minimum total MEJ gap reduces with increasing wave speed.

In the range of lower frequency ratios there is no clear tendency of the influence of the wave apparent velocity. Higher wave speed does not necessarily cause smaller required gap. This is because spatially varying ground motions induce both quasi-static and dynamic responses. When a structure is relatively stiff, quasi-static response dominates the responses. That is why when the frequency ratio is larger than 1.15, which is achieved by increasing the stiffness of the right bridge structure while that of the left bridge structure remains unchanged, increasing wave apparent velocity reduces the relative responses. On the other hand, if the structure is relatively flexible, dynamic response dominates, which is very much influenced by the dynamic properties of the bridge and the frequency content of the ground motions. These observations indicated that the minimum gap is not only significantly affected by the frequency ratio of two adjacent structures, but also by the absolute vibration frequencies of the two structures.

A comparison of the results with and without SSI shows that an additional effect of SSI further increases the minimum gap that a MEJ must have to ensure that pounding will not take place. This result clearly shows the significance of the combined influence of SSI, the spatial variation of the ground excitations and the ratio of the fundamental frequency of the adjacent bridge structures.

4.2 Opening relative movements

To ensure a perfect function of MEJ each of the seals between the MEJ beams should not be overstretched during the opening relative movement between the bridge girders. In order to achieve this goal MEJ must be designed with sufficient number of centre beams so that it can cope with the largest opening relative movement without causing any damage to the covering seals.

Figure 6 shows the combined influence of SSI and the spatial variation of the ground excitations on the mean values of the largest expected opening movement g_o between the bridge girders. If a uniform ground excitation is assumed, theoretically almost no opening relative movement will take place, because both adjacent structures have nearly the same fundamental frequencies ($f_{II}/f_I = 0.99$). However, because the adjacent structures have different heights (Figure 2), the subsoil has not the same influence on each of the structures as it can be easily estimated from Equation (1). Consequently, the unequal soil-structure interaction causes opening relative movement g_o of 10.4 cm, even if both structures experience the same ground excitation. In Figure 6 this result is displayed as a solid horizontal line. It clearly shows the significance of the unequal soil-structure interaction effect.

If the influence of the wave apparent velocity is considered as well, the largest opening relative girder movement increases to 12.5 cm in the case of c_a = 500 m/s. Similar to the total gap that a MEJ requires to avoid pounding, the relative movement between the girders does not increase proportionally with the wave speed which indicates that other influence is also strongly involved.

The results in the case of a constant wave velocity c_a = 500 m/s confirms the involvement of many factors in the development of the relative movement between the girders. In contrast to the expectation the largest opening relative movement g_o of 12.5 cm occurs, when the spatially varying ground excitations have the highest correlation. Weakly correlated ground motions do not cause the largest opening relative girder movement, but the value of 11.75 cm, while the intermediately correlated ground motions produce a smaller opening relative movement of 10.72 cm.

To focus on the combined influence of SSI, the spatial variation of the ground motions and the ratio of the fundamental frequencies of the adjacent structures, it is assumed that both bridge structures have the same height of 9 m and the ground motions are highly correlated.

Figure 7 shows this combined effect on the largest opening relative movement that a MEJ should expect. If SSI effect is neglected, the apparent velocity of the spatially varying ground motions is dominant when the frequency ratio f_{II}/f_I is larger than 1. Even though the opening relative movement in the higher frequency ratio range does decrease with the apparent velocity, similar to the total gap required to avoid girder pounding, the smallest opening relative movement does not occur when both adjacent structures have the same fundamental frequency (f_{II}/f_I = 1).

An additional consideration of SSI alters the result significantly. In general it can be observed that SSI amplifies in almost all cases the opening relative movement.

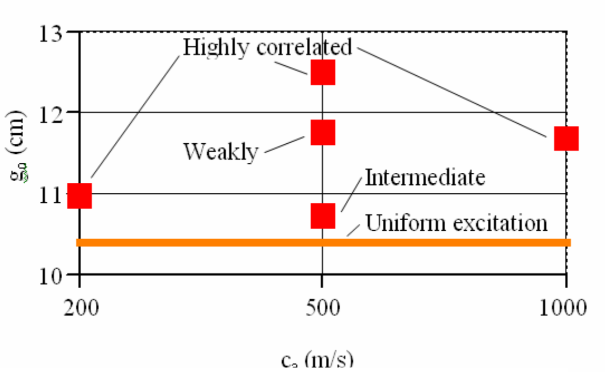

Figure 6 Minimum opening movability g_o of a MEJ.

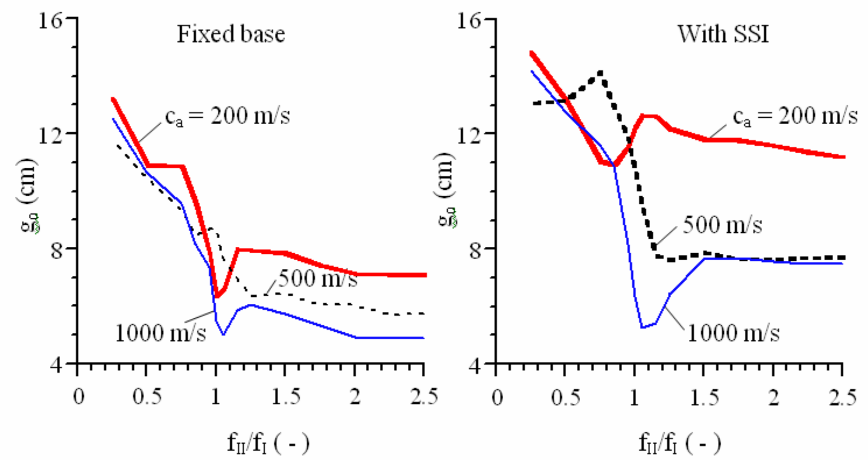

Figure 7 Dependence of the minimum required opening movability g_o on the frequency ratio f_{II}/f_I, the apparent wave velocity c_a of the spatially varying ground motions and SSI.

While in the higher frequency ratio range above 1.5, an apparent velocity c_a higher than 500 m/s does not further reduce the opening relative movement which remains almost constant around 7.6 cm. At lower frequency ratios the difference in the fundamental frequencies of the adjacent bridge structures has significant influence on the development of the largest opening relative movement that one should consider in the design of MEJ. The result with SSI effect also shows that current design recommendation by using only the frequency ratio f_{II}/f_I as the design parameter is clearly insufficient.

For the design of MEJ both values, the minimum total gap required and the largest opening relative movement, are essential. The number of the middle beams of MEJ should be chosen so that the total available gaps between MEJ beams and the total available seals covering the gaps can cope with the largest opening and closing relative movement between the bridge girders.

5 CONCLUSIONS

A new design philosophy for preventing bridge girders from pounding due to strong earthquake is introduced. In contrast to the design of a conventional bridge expansion joint with only a few-centimetre gap, in the new design, modular expansion joints (MEJs) are installed so that the adjacent bridge girders can have a large relative movement without causing any pounding, and consequently damage to the girders. The most significant specification is the minimum total gap of the joint. The MEJs should then be designed so that the total MEJ gap can cope with the largest expected closing relative movement. The other significant specification is the largest opening relative movement that the MEJs can expect. They must be designed so that they can cope with these movements without causing any damages to the seals between the MEJ beams.

In this work the influence of the spatially varying ground motions, SSI and their combined effect are discussed.

The investigation shows that:

- The recommendation of current design regulations to adjust the fundamental frequencies of adjacent bridge structures does not necessarily produce the smallest minimum total gap that a MEJ must have when the ground motions are not uniform and the soil is soft.
- When the frequency ratio of adjacent bridge spans is larger than 1.15, the effect of wave apparent velocity dominates the relative response. The minimum required gap decreases, as expected, with higher wave speed.
- In the lower frequency ratio range the combined effect of the ground motion spatial variation, SSI and the vibration frequencies of the adjacent structures governs the minimum required gap.
- In almost all cases SSI causes a larger total gap.

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