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Numerical investigation of seismic gap between adjacent structures with SFSI

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ABSTRACT

Relative response can cause pounding of adjacent structures. In current seismic design specifications, structural pounding should be avoided by designing structures of similar fundamental frequencies and by having sufficient seismic gap. In reality, however, the relative response between adjacent structures is also affected by the soil. During a strong earthquake the supporting soil can deform and interact with the structures. The response of structure with structure-foundation-soil interaction (SFSI) will be different with that of structures with an assumed fixed base. Neglecting SFSI effect can result in an insufficient seismic gap between structures and thus cause pounding damage to structures. In this study, the seismic gap requirement between structures with different support conditions was investigated. A macro-element model was utilized to simulate a plastic deformation of soil. Stochastically simulated earthquake based on Japanese design spectrum was applied as an excitation of two adjacent structures with different mass and stiffness distribution. Numerical result indicates that SFSI can significantly alter the relative response of adjacent structures. The consequence of SFSI for the seismic gap requirement of adjacent structures will be discussed.

1. INTRODUCTION

Pounding of adjacent structures can cause damage in the structure. It has been observed and reported follow by many major earthquake events (Cole et al., 2011, Chouw and Hao, 2012, Palermo et al., 2011). Pounding takes place when the closing relative response exceeds the seismic gap between the structures. To minimize the relative response adjacent structures should be designed so that they have the same of similar fundamental frequencies. In reality, however, the ground is nonuniform. Consequently, the relative response is often unavoidable even if the same ground excitation can be justified. In the case of long extended structures, e.g. pipe system or

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long bridge a spatial variation of the ground excitation can strongly influence the development of the relative response between adjacent structural members (Bi et al. 2010, 2011, 2013, Chouw and Hao, 2004, 2008, 2009). The number of the influence factors can be further increased if the structures interact with the subsoil unequally (Chouw, 2002, 2008). However, most studies were performed numerically. Experimental investigations on the effect of spatially varying ground motions on relative responses of adjacent structures are very limited (e.g. Li et al. 2012, 2013). Qin and Chouw (2010) have performed an experimental study, revealing that plastic deformation of foundation soil is very likely to take place under the strong seismic response of structure. This nonlinear SFSI can increase structural response and thus cause pounding with adjacent structures.

The effect of SFSI on the relative response between structure and structural pounding has been studied using a number of numerical approaches. Chouw and Hao (2008a) used a boundary element formulation to simulate the response of bridge structure with linear soil behavior. They have discussed the effect of pounding force including SFSI. Using the same formulation, they have also evaluated the minimum total gap that a modular expansion joint (MEJ) must have to prevent pounding between bridge girders (Chouw and Hao, 2008b). In their study, non-uniform ground excitation was considered. However, study in the past was conducted with the assumption that the soil behavior was linear. Research using nonlinear SFSI was only focus on its effect on the structural response. Qin et al. (2013) has investigated the interaction of nonlinear soil with plastic hinge development in the structure. Paolucci (1997) has proposed a formulation for calculating the response of structure with nonlinear SFSI. The accuracy of the proposed formulation was validated using shake table test (Paolucci et al. 2008). However, the effect of nonlinear SFSI on the relative response between structures has not been reported.

In this study, the seismic gap between two structures with possible plastic deformation of soil was investigated. Nonlinear time history analysis was conducted using excitation simulated based on Japanese design spectrum (JSCE, 2000). The foundation soil nonlinearity was simulated using a macro-element model. While the dynamic property of the first building was fixed, the mass and stiffness distribution of the second building was varied to consider the different frequency ratio of the adjacent structures. At the beginning the relative response between structures with the same fundamental frequency is investigated. However, the stiffness and mass distribution of the structures were different. In the following investigation the effect of mass ratio of adjacent structures on their seismic gap requirement is considered. This step was done by assuming the stiffness distribution of the two buildings was identical. Only the seismic mass was altered. For each step, structures with a fixed base and with nonlinear soil were considered.

2. NONLINER TIME HISTORY ANALYSIS

2.1 Structural model

Two structures, S1 and S2, were considered to estimate the seismic gap required to avoid pounding (Fig. 1). Both structures were four-story steel building and had a similar layout with a floor area of 6.7 m x 6.7 m. The inter-storey high was 3 m. The beam and column of S1 were constructed using 410UB53.7 and 310UC118. Based on New Zealand Design Standard (NZS 1170.5, 2004 and NZS 3404, 1986), the seismic masses of the building were 30 tones and 25 tones for the floor and roof levels, respectively. The fundamental frequency of S1 was 1.56 Hz. Because this study focused on the effect of soil nonlinearity, for simplicity it is assumed that S1 and S2 can be represented by their fundamental mode using a SDOF structures (M1 and M2). The effective mass and high of the M1 was obtained based on the mass and stiffness distribution of S1. The lateral stiffness of the structure was obtained by matching its fundamental frequency with the one of S1.

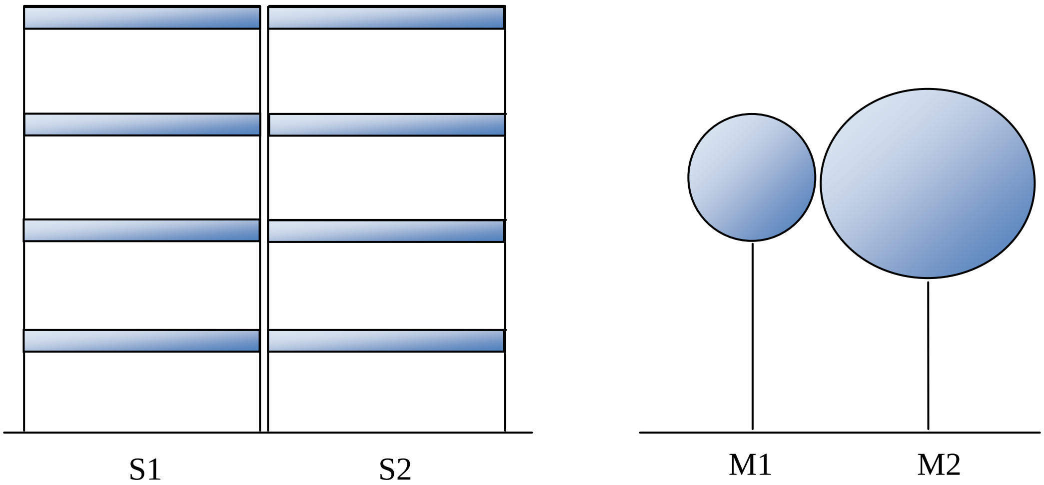


Fig. 1 Structures and their equivalent SDOF structures

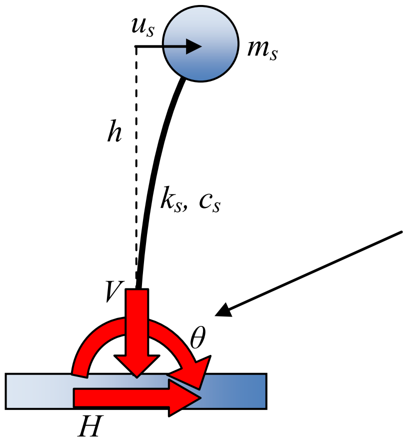
The SDOF model (M2) was utilized to represent the fundamental mode of the adjacent structure (S2). While the dynamic property of M1 was kept constant, the stiffness and mass of M2 was varied. The relative response between M1 with various M2 was investigated. Three different cases were performed in this study. The considered mass of M2 was given in Table 1. The first two cases were conducted to evaluate the current design specifications, i.e. recommending adjacent structures of the same or very similar fundamental frequencies so that out-of-phase relative movements can be minimized or eliminated. This recommendation can be followed in the case of an assumption that structures are fixed at their bases. In reality, however, this is not feasible when the soil and foundation response are considered. Because the response of adjacent structures depends not only on the dynamic properties of the structures but also on the interaction between structures, foundations and subsoil, eliminating structural pounding by only matching the fundamental frequencies of adjacent structures is insufficient. Therefore, in the Cases 1 and 2, the seismic mass and stiffness of M2 was varied to be ± 30% of the corresponding M1's properties. The fundamental frequency of M2 will be the same with that of M1. The current design recommendation for avoiding pounding with the adjacent structure can then be evaluated. On the other hand, even though adjacent structures were designed to be identical and constructed using the same structural element, the actual mass in a building can be different. Thus, Case 3 was considered to reveal the effect of mass ratio on the relative response in adjacent structure. The stiffness of M2 and M1 was assumed to be the same. The top mass of M2 was altered so that it is 10% smaller than that of M1. In this case, the frequency ratio between structures was 1.056.

Table. 1 Property of M1 and M2 and their frequency ratio in different cases

Case	Mass M1 (t)	Mass M2 (t)	f_{M1}/f_{M2}
1	92	64.4	1
2	92	119.6	1
3	92	82.2	1.056

2.2 Macro-element model

The response of a structure with nonlinear SFSI was calculated with the help of a macro-element model. The concept of the model is to simplify the response of a foundation-soil system using three dynamic degrees-of-freedom (3DOF). They are the vertical, horizontal displacements and rotation of the center of the foundation. With an additional DOF which is representing the horizontal top displacement of the structure, the response of the structure-foundation-soil system can be calculated using a 4DOF model. Fig. 2 shows the macro-element model. When the soil behaves linearly, the stiffness matrix of the 3DOF foundation system can be calculated using the formulation described in Chouw and Hao (2008a). By combining the matrix for the degrees of freedom of the foundation system (Fig. 2) with the horizontal degree of freedom of the structure, the equation of motion for the whole structure-foundation-soil system can be derived (Eq. 1).



$$\mathbf{m_F} = \begin{bmatrix} m_s + m_o & m_s h & 0 \\ m_s h & m_s h^2 + J_o & 0 \\ 0 & 0 & m_s + m_o \end{bmatrix}$$

$$\mathbf{k_F} = \begin{bmatrix} k_H & & \\ & k_\theta & \\ & & k_V \end{bmatrix} \quad \mathbf{c_F} = \begin{bmatrix} c_H & & \\ & c_\theta & \\ & & c_V \end{bmatrix}$$

$$\mathbf{u_F} = [H \quad \theta \quad V]^T$$

Fig. 2 Four DOF macro-element model

$$\begin{bmatrix} m_s & m_s & m_s h & 0 \\ m_s & & & \\ m_s h & & \mathbf{m_F} & \\ 0 & & & \end{bmatrix}\begin{bmatrix} a_s \\ \\ \mathbf{a_F} \\ \\ \end{bmatrix}+\begin{bmatrix} c_s & 0 & 0 & 0 \\ 0 & & & \\ 0 & & \mathbf{c_F} & \\ 0 & & & \end{bmatrix}\begin{bmatrix} v_s \\ \\ \mathbf{v_F} \\ \\ \end{bmatrix}+\begin{bmatrix} k_s & 0 & 0 & 0 \\ 0 & & & \\ 0 & & \mathbf{k_F} & \\ 0 & & & \end{bmatrix}\begin{bmatrix} u_s \\ \\ \mathbf{u_F} \\ \\ \end{bmatrix}=P \tag{1}$$

2.3 Nonlinearity of the foundation system

The nonlinearity of the structure-foundation-soil system is concentrated at the 3DOF of the foundation system. The nonlinearity will be initiated when the bearing failure of soil has taken placed. To determine the initiation of the bearing failure, a bearing strength surface was considered. Nova and Montrasio (1991) has proposed an equation for determining the initiation of bearing failure of strip footing on sand (Eq. (2)). The equation was developed and improved though a number of experimental and numerical studies (e.g. Paolucci et al., 2008).

$$f(\mathbf{F})=\left(\frac{H}{\mu V_{\max}}\right)^2+\left(\frac{M}{\psi B V_{\max}}\right)^2-\left(\frac{V}{V_{\max}}\right)^2\left(1-\frac{V}{V_{\max}}\right)^{2\xi} \tag{2}$$

where B is the width of the foundation; V_{max} is the ultimate bearing capacity of foundation soil under vertical centered load; ψ , μ and ξ are the parameters of the bear strength surface and suggested to be 0.43, 0.9 and 0.95, respectively (Paolucci et al., 2008); $\mathbf{F}$ is a vector which contains the horizontal, vertical actions and moment at the foundation (*H*, *V* and *M* in Fig. 2).

With $\mathbf{F}$ calculated using the product of the stiffness matrix ($\mathbf{k_F}$) and the displacement vector ($\mathbf{u_F}$) of the foundation. Eq. (2) can be evaluated. If $f(\mathbf{F})$ < 0, the combined action of shallow foundation is below the bearing capacity of foundation soil. Bearing failure of soil does not occur. On the other hand, a value of $f(\mathbf{F})$ = 0 indicates that the foundation action will cause bearing failure. Theoretically, it is impossible to have a value of $f(\mathbf{F})$ > 0, since the soil cannot sustain any actions greater than the bearing capacity.

Once the indication of bearing failure was confirmed, a non-associated flow rule (Eq. (3)) was applied to calculate the unrecoverable displacement and rotation of the foundation due to soil plastic deformation.

$$g(F)=\lambda^2\left(\frac{H}{\mu V_{\max}}\right)^2+\chi^2\left(\frac{M}{\psi B V_{\max}}\right)^2+\left(\frac{V}{V_{\max}}\right)^2-1 \tag{3}$$

where B is the width of the foundation; λ and χ are two non-dimensional parameters, and V_{max} is the maximum bearing capacity of soil under vertical loading.

This flow rule was originally developed by Cremer et al. (2001). Negro et al. (2000) and Paolucci et al. (2008) have conducted a large scale shake table test to quantify the non-dimensional parameters of Eq. (3). For sand, the values of λ and χ were suggested to be 2.5 and 3, respectively. In this study, the supporting soil was defined to be sand and had a shear wave velocity of 100 m/s. The Poisson's ratio and density were assumed to be 0.33 and of 2000 kg/m^3, respectively. It is assumed that the structure behaves elastically during the earthquake loading. The soil condition of the structures was also assumed to be the same.

2.4 Earthquake excitation

The ground excitation for the nonlinear time history analysis was simulated based of Japanese design spectrum (Chouw and Hao, 2008b). In this study, medium soil category was considered. This is because in general, loose soil is of no practical interest for shallow foundation design. An alternative foundation scheme, such as foundation through piles, is usually preferred. On the other hand with dense or very-dense soil condition significant settlements are not likely to occur (Paolucci, 1997). The applied ground acceleration is shown in Fig. 3.

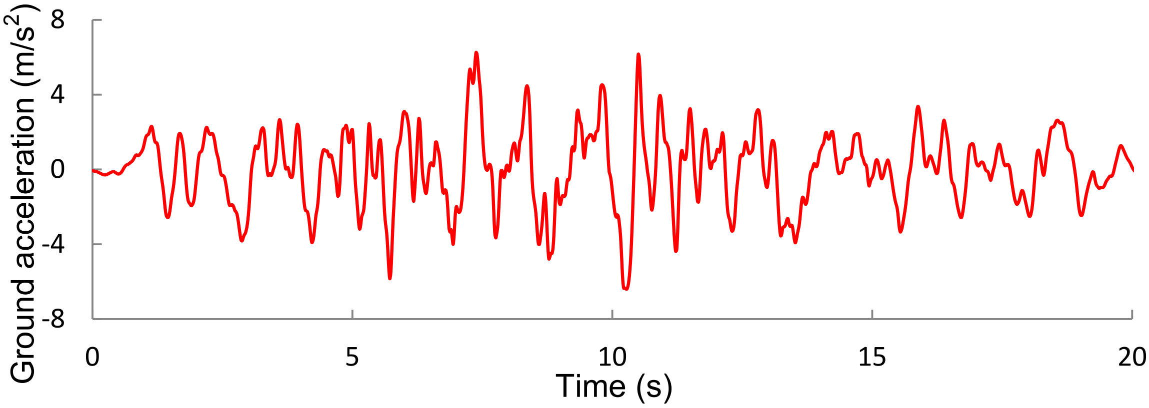


Fig. 3 Ground acceleration

3. EFFECT OF LINEAR SFSI ON THE SEISMIC GAP

Fig. 4 shows the time history of horizontal displacement (*u*) at the top of the structures. The structures were having the same fixed base natural frequency but different stiffness and mass distribution. While Fig. 4(a) illustrated the response of M1 and M2 of Case 1, Fig. 4(b) showed that of Case 2. Linear soil is assumed. The results clearly show when soil deformation is permitted, the relative displacements are different even though both cases 1 and 2 have the same fixed-base fundamental frequency ratio f_{M1}/f_{M2} of 1. The result indicates that the difference in their distribution of mass and stiffness altered the actual SFSI. Consequently, the seismic responses of M1 and M2 with soil effect are different. The results suggest that the fixed-base fundamental frequency of structure cannot be used as the only factor for eliminating pounding between adjacent structures. In the considered case of an assumed elastic soil, the seismic gaps required to avoid pounding are 75.8 mm and 78.4 mm for cases 1 and 2, respectively.

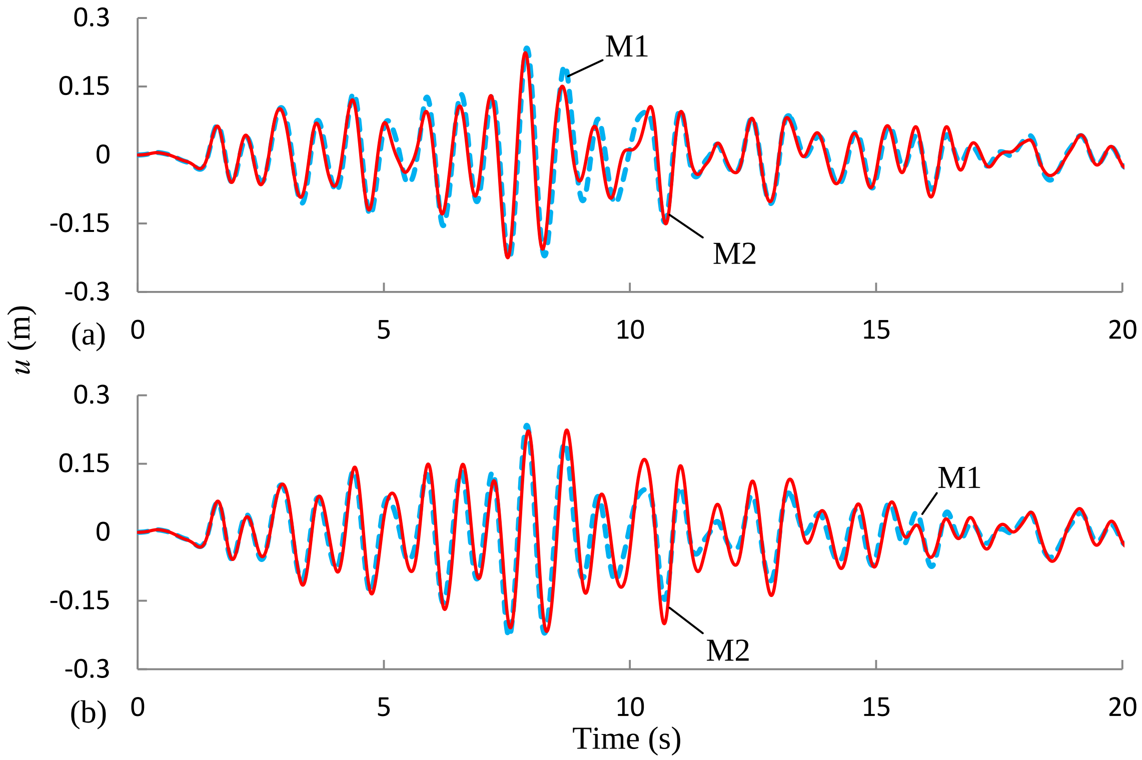


Fig. 4 Horizontal displacement at the top of structures in Cases (a) 1 and (b) 2 with elastic SFSI

4. EFFECT OF NONLINEAR SFSI ON THE SEISMIC GAP

Fig. 5 showed the relative horizontal displacement (*u*) with nonlinear SFSI. While Fig. 5(a) shows the response of Case 1, Fig. 5(b) illustrated that of Case 2. Because the mass and stiffness distribution of structures is different, these responses are not the same. The nonlinear SFSI can reduce the relative response between adjacent structures. With nonlinear soil behavior, the maximum relative response between M1 and M2 is 50.3 mm and 41.4 mm for cases 1 and 2, respectively. They are 33.6% and 47.2% smaller than those with elastic soil assumption. It shows that designing a structure with fixed-base assumption can underestimate the pounding potential between adjacent structures. If during an earthquake deformation of soil occurs, pounding is more likely to take place. However, nonlinearity of soil can reduce the seismic gap requirement for adjacent structures.

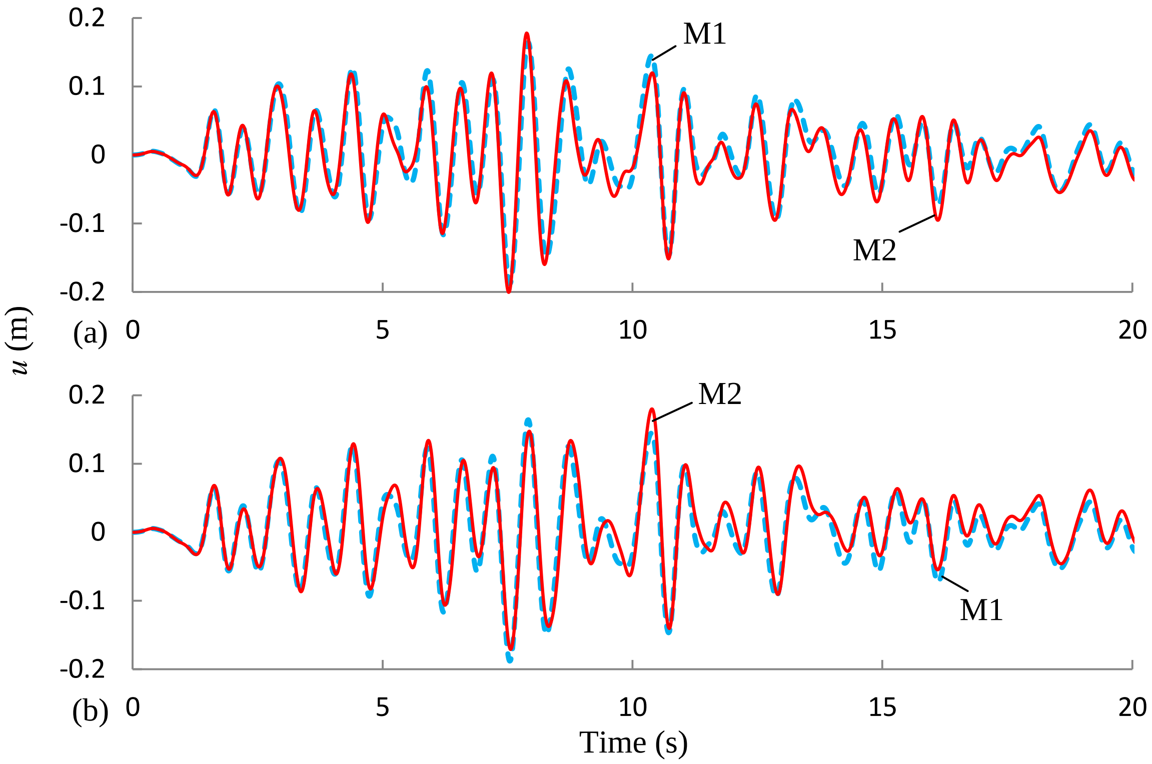


Fig. 5 Horizontal displacement at the top of structures of Case (a) 1 and (b) 2 with nonlinear SFSI

5. SEISMIC GAP OF ADJACENT STRUCTURE WITH DIFFERENT SEISMIC MASS

In practice, adjacent structures will likely have different mass. Although they might be designed and constructed with the same structural element, their fundamental frequency will be different. Hence, large relative response might occur and consequently pounding can take place. In this study, the stiffness distribution of M1 and M2 was assumed to be identical. The top mass of M2 was verified as demonstrated in Case 3 (Table 1). Fig. 6 shows the horizontal displacement (*u*) at the top of the structures with an assumed fixed base. Although the stiffness and support condition of both structures are the same, the seismic mass is 10% different. This produced a 5.13% difference in their fixed base natural frequencies. During an earthquake, the maximum lateral relative displacement (*u*) of M1 was larger than that of M2. The maximum relative movement between M1 and M2 was 89.5 mm, suggesting that pounding could take place during earthquake if seismic gap is not provided between structures. On the other hand, when plastic deformation of soil is permitted (Fig. 7), the relative response between M1 and M2 was reduced to 75.1 mm, i.e. a 16.1% reduction.

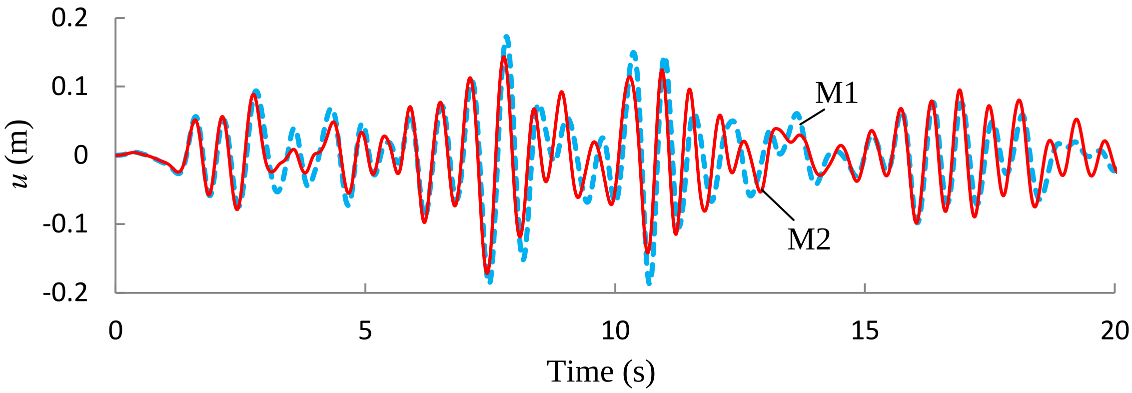


Fig. 6 Horizontal displacement at the top of structures with a fixed base

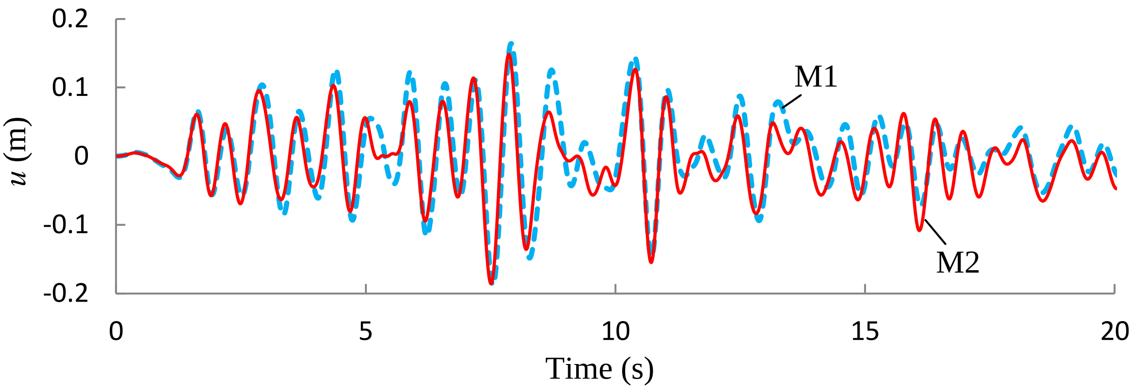


Fig. 7 Horizontal displacement at the top of structures with nonlinear SFSI

6. CONCLUSIONS

In this study, the required seismic gap between adjacent structures with nonlinear SFSI was investigated. Nonlinear time history analysis was conducted using excitation simulated based on Japanese design spectrum. The nonlinearity of the footing-soil system was simulated using a macro-element model. Two buildings with different properties were considered. While the dynamic property of one of the structures was fixed, the mass and stiffness distribution of the neighbouring structure was varied. Three cases were considered, the first two cases involved in simulating the relative response between structures with the same fundamental frequency but of different structural properties. The third case considered the effect of mass ratio of adjacent structures on their seismic gap requirement. It is assumed that the stiffness distribution of the two buildings was identical. Only the seismic mass was altered. For each case, structures with a fixed base and nonlinear soil were considered.

The investigation reveals that:

- When plastic soil deformation was permitted, SFSI depends on the stiffness and mass distribution of the structure. Therefore, even if adjacent structures have very similar fundamental frequencies, pounding can still take place.

- In the considered cases, when the lateral stiffness of adjacent structures was the same, a 10% difference between the mass of structures can cause pounding. However, nonlinear SFSI interaction can reduce the relative displacement between structures.

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