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A PARAMETRIC STUDY FOR THE DYNAMIC RESPONSE CONTROL OF ADJACENT BUILDINGS

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ABSTRACT

The purpose of this paper is to find out if the response amplitude of two adjacent buildings which collide during an earthquake can be controlled by the interposition of unilateral visco-elastic gap elements. To this end the dynamical analysis of two adjacent planar multi-storey shear-type frames, subjected to seismic excitations, has been performed. Firstly a numeric procedure for solving the nonlinear equations of motion, which allows partially elastic impacts as well as visco-elastic coupling, is presented. Secondly some numerical applications under harmonic excitations are reported, showing that the dynamical characteristics of both systems are considerably changed by pounding. Finally a parametric study points out that, with a suitable choice of the gap element properties, the interaction between colliding structures can be exploited to control the amplitude of their structural response.

KEYWORDS

Passive control; pounding; unilateral constraints; unilateral visco-elastic gap elements; elastic and partially elastics impacts; nonlinear systems; seismic behaviour of adjacent colliding buildings.

INTRODUCTION

Pounding between adjacent buildings due to earthquakes can be one of the causes of relevant or severe structural damage. Reports on recent strong earthquakes (Bertero, 1986; Kasai *et al.*, 1992) show that damages as a result of pounding can be divided into two groups: local damage at the contact area of adjacent buildings and severe structural damage in which either partial or total failure occurs. It should be noticed that pounding took place in approximately 40% of the heavily damaged or collapsed buildings during the severe Mexico City earthquake in 1985. To avoid such inconveniences, modern seismic codes require a sufficiently large separation between neighbouring buildings in order to enable each one to vibrate freely without contact. However most of the existing constructions do not respect such provisions so they should be considered risky. Moreover code provisions lead to building gaps that are inconsistent with the code philosophy according to which large inelastic deformation can occur during major earthquakes.

Due to their central importance in seismic behaviour of adjacent buildings, pounding effects have been recently investigated by many researchers from both a theoretical and an experimental point of view. In some studies pounding has been modeled by unilateral gap elements which connect the two neighbouring dynamic systems (Anagnostopoulos, 1988; Liolios and Galouissis, 1992; Maison and Kasai, 1990; Kasai *et al.*, 1992; Spiliopoulos and Anagnostopoulos, 1992). In other studies the impact forces have been determined imposing the conservation of total energy and momentum of the colliding bodies, evaluating the energy dissipated by means of an appropriate coefficient of restitution (Athanassiadou *et al.*, 1994; Conoscente *et al.*, 1992; Papadrakakis *et al.*, 1991). Some experimental tests have also made it possible to estimate consistent values of the coefficient of restitution from measurements of pre- and post-impact velocities (Leibovich *et al.*; 1994; Papadrakakis *et al.*, 1995).

Resuming the results of a previous work (Pasquino *et al.*, 1995), the aim of the present paper is to investigate if and when it is possible to control the amplitude of the dynamical response of colliding buildings by interposing unilateral gap elements which consist of an elastic spring and a viscous dash-pot. Such devices can, therefore, be considered as a particular type of passive control system for structural vibrations.

For this purpose the dynamical analysis of a planar model, consisting of two shear-type frames linked together by these elements at corresponding floors, has been performed. Each gap element is activated when the relative displacement of the two adjacent floors equals the gap size. As long as the relative displacement of at least one floor is larger than the gap size, the two frames vibrate together as a coupled system. The equations of motion are, therefore, piecewise linear and the dynamical response has been determined by a numerical method. A special technique has been developed, which has the peculiar feature to consider a partially elastic impact when the relative displacement equals the gap size and the spring shortening.

Some numerical investigations have been carried out under harmonic excitations in order to elucidate the main parameters which govern the dynamical behaviour of the system. The results, reported in the form of frequency response functions, show that the presence of unilateral constraints may determine a reduction or an amplification of the structural response. The analyses under harmonic excitations point out how the interaction of the two buildings strongly modifies the frequency response of both systems and show that it is sometimes possible to control the response of both systems through the interposition of appropriate dissipative gap elements, leading to a significant reduction of the structural response.

THE STRUCTURAL MODEL

The structural model, Fig. 1, is composed of two adjacent planar shear-type frames with a different number of floors, n_1 and n_2. Both frames have the same interstorey height and an unilateral visco-elastic gap element is interposed between any two corresponding floors.

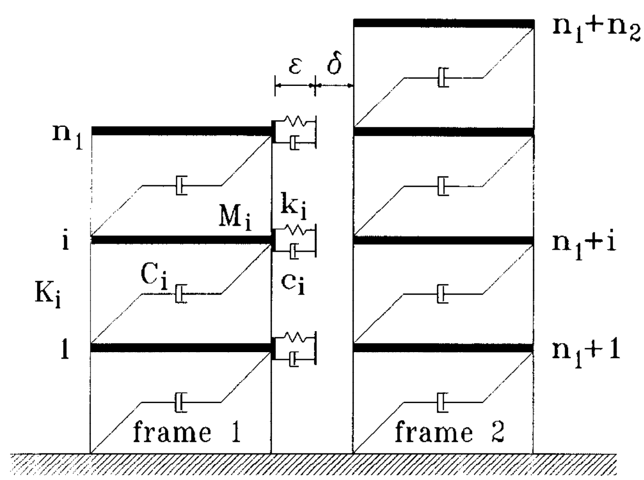

Fig. 1. The structural model.

The dynamical degrees of freedom are the floor translations with respect to the ground, arranged as shown in Fig. 1. The mass of the i-th floor and the stiffness and damping coefficients of the underlying interstorey are indicated with M_i, K_i and C_i respectively. The stiffness and the damping coefficients of the gap element between the i-th and the n_1+i-th floor are denoted with k_i and c_i. The gap size δ and the allowable spring shortening ε are assumed constant for all the gap elements.

EQUATIONS OF MOTION

The equations of motion for the system under seismic excitations are written as follows

$$\mathbf{M}\ddot{\mathbf{v}} + \mathbf{C}(t)\dot{\mathbf{v}} + \mathbf{K}(t)\mathbf{v} = -\mathbf{M}\mathbf{1}\ddot{v}_g(t) + \mathbf{a}(t) \tag{1}$$

where $\mathbf{v}$ is the vector of the $n_1 + n_2$ degrees of freedom;
$\mathbf{M}$ is the mass matrix;
$\mathbf{C}(t)$ is the damping matrix at time t;
$\mathbf{K}(t)$ is the stiffness matrix at time t;
$\mathbf{1}$ is a unit vector;
$\ddot{v}_g(t)$ is the free-field ground acceleration;
$\mathbf{a}(t)$ is a vector, of which the components depend on the effective unilateral constraints at time t.

The system of equations (1) is nonlinear because the damping matrix $\mathbf{C}(t)$, the stiffness matrix $\mathbf{K}(t)$ and the vector $\mathbf{a}(t)$ depend on the number and the position of the unilateral constraints which are in contact with both frames at time t. The mass matrix is constant and has a diagonal form, with the coefficients equal to the storey masses. When there is no coupling between the two frames, i.e. no unilateral constraint is effective, $\mathbf{a}(t) = \mathbf{0}$ and the damping and stiffness matrices take the form

$$\mathbf{C} = \begin{bmatrix} \mathbf{C}_1 & \mathbf{0} \\ \mathbf{0} & \mathbf{C}_2 \end{bmatrix}, \qquad \mathbf{K} = \begin{bmatrix} \mathbf{K}_1 & \mathbf{0} \\ \mathbf{0} & \mathbf{K}_2 \end{bmatrix},$$

where $\mathbf{C}_1$, $\mathbf{C}_2$, $\mathbf{K}_1$ and $\mathbf{K}_2$ are respectively the tri-diagonal damping and stiffness matrices of both frames. Two adjacent floors, corresponding to the degrees of freedom i and $j = n_1 + i$, interact when their relative displacement is greater than the gap size δ, i.e. when

$$v_i - v_j - \delta > 0. \tag{2}$$

In this case a force F_i, expressed by the equation

$$F_i = c_i(\dot{v}_i - \dot{v}_j) + k_i(v_i - v_j - \delta), \tag{3}$$

arises between the two interacting floors and some of the coefficients of matrices $\mathbf{C}(t)$ and $\mathbf{K}(t)$ and of vector $\mathbf{a}(t)$ are modified as follows

$$C_{ii}(t) = C_{ii} + c_i; \quad C_{jj}(t) = C_{jj} + c_i; \quad C_{ij}(t) = C_{ji}(t) = -c_i;$$

$$K_{ii}(t) = K_{ii} + k_i; \quad K_{jj}(t) = K_{jj} + k_i; \quad K_{ij}(t) = K_{ji}(t) = -k_i;$$

$$a_i(t) = k_i\delta; \qquad a_j(t) = -k_i\delta.$$

When the floors corresponding to the degrees of freedom i and j cease to interact, these coefficients take their initial values again.

SOLVING THE EQUATIONS OF MOTION

The Newmark numerical integration method has been used for solving the nonlinear equations of motion. It has been set $\gamma = 1/2$ and $\beta = 1/4$, which ensures the unconditional stability of the solution. A particular attention has been deserved to the evaluation of the contact and separation times of two adjacent floors, in which the matrices $\mathbf{C}(t)$ and $\mathbf{K}(t)$ and the vector $\mathbf{a}(t)$ have to be modified. These times have been determined, within an accepted tolerance, by an iterative procedure based on bisection of time step amplitude Δt.

Impact and post-impact conditions

When the relative displacement between two adjacent floors equals the initial gap size δ and the allowable spring shortening ε, an impact occurs. This condition, referred to floors corresponding to the degrees of freedom i and j, leads to the equation

$$v_i - v_j - \delta - \varepsilon = 0. \tag{4}$$

The impact times have been evaluated by the same iterative procedure utilized for the contact and separation times. In this case the equations of motion do not change and the dynamical response depends on the dissipated energy. This has been determined by means of a coefficient of restituition e, assuming a partially elastic impact. The post-impact velocities have been evaluated according to the impact law, imposing the conservation of momentum (Conoscente *et al.*, 1992)

$$\dot{v}_i^+ = \frac{M_i - e\,M_{n_1+i}}{M_i + M_{n_1+i}}\,\dot{v}_i^- + \frac{(1+e)\,M_{n_1+i}}{M_i + M_{n_1+i}}\,\dot{v}_{n_1+i}^-$$

$$\dot{v}_{n_1+i}^+ = \frac{(1+e)\,M_i}{M_i + M_{n_1+i}}\,\dot{v}_i^- + \frac{M_{n_1+i} - e\,M_i}{M_i + M_{n_1+i}}\,\dot{v}_{n_1+i}^- .$$

NUMERICAL APPLICATIONS

The numerical applications reported herein have the purpose to illustrate how the pounding between two adjacent buildings may be exploited to control their response amplitude. In order to limit the number of parameters which govern the behaviour of the structural system, two periodic frames have been considered. This assumption is almost realistic and allows simple parametric studies without loss of generality. In the following the two frames will be termed frame 1 and frame 2 and their mass, damping and stiffness properties, constant for all the floors, will be indicated by M_1, C_1, K_1 and M_2, C_2, K_2 respectively. The damping and stiffness characteristics of the unilateral gap elements, c and k, have been also considered constant for any couple of corresponding floors. In this way the structural system may be described by the following nine parameters

$$\lambda^2 = \frac{K_1}{M_1},\quad \alpha = \frac{K_2}{K_1},\quad \beta = \frac{M_2}{M_1},\quad \gamma = \frac{C_2}{C_1},\quad \bar{\xi} = \frac{C_1}{2M_1\lambda},\quad \bar{\delta} = \frac{\delta}{g}\lambda^2,\quad \eta = \frac{k}{K_1},\quad \mu = \frac{c}{M_1\lambda},\quad \bar{\varepsilon} = \frac{\varepsilon}{g}\lambda^2.$$

where g is the gravity and the other symbols have been previously defined. The excitation is provided by a harmonic free-field ground acceleration

$$\ddot{u}_g(t) = \ddot{u}_{go}\exp\{i\bar{\omega}t\}$$

acting simultaneously at the base of both frames. Dynamic soil-structure interaction has not been taken into account, so that the free-field motion coincides with that experienced by the base of the frames.

Three different structural systems have been taken into account, all subjected to a base acceleration of amplitude 0.2 g. The first is composed of two single storey frames. Despite its simplicity, this system enables to point out the main aspects of the frequency response of two buildings having the same height. Frame 1 is stiffer than frame 2, the ratio between their fundamental periods is equal to two and their damping ratios are both equal to 5% when they vibrate without interaction. In Fig. 2 the amplitude of the frequency response function of the dimensionless shear force at the base is reported against the frequency ratio β, where $\bar{\omega}$ and ω_1 are the frequencies of the excitation and of frame 1 respectively and W is the weight of frame 1. In the same figure the frequency response functions of the two frames freely vibrating without interaction are also reported for comparison. The graphs of Fig. 2a refer to the case of elastic impacts between the two frames, i.e. when no gap element is interposed and the coefficient of restitution e is set at one. As it may be noticed, the curves related to the interacting frames exhibit two peaks which are in a different position in comparison with the peaks of the curves of the two independent frames. Moreover the stiffer interacting frame shows a remarkable increase of the higher peak amplitude. On the contrary the more flexible interacting frame shows a decrease of the same quantity. From this point of view the stiffer frame is more stressed because of interaction, while the more flexible is less stressed. However it should also be noticed that there exists a range of frequencies, between the resonances of the two independent frames, where both curves exhibit an increased amplitude in comparison with the curves of the free vibrating frames.

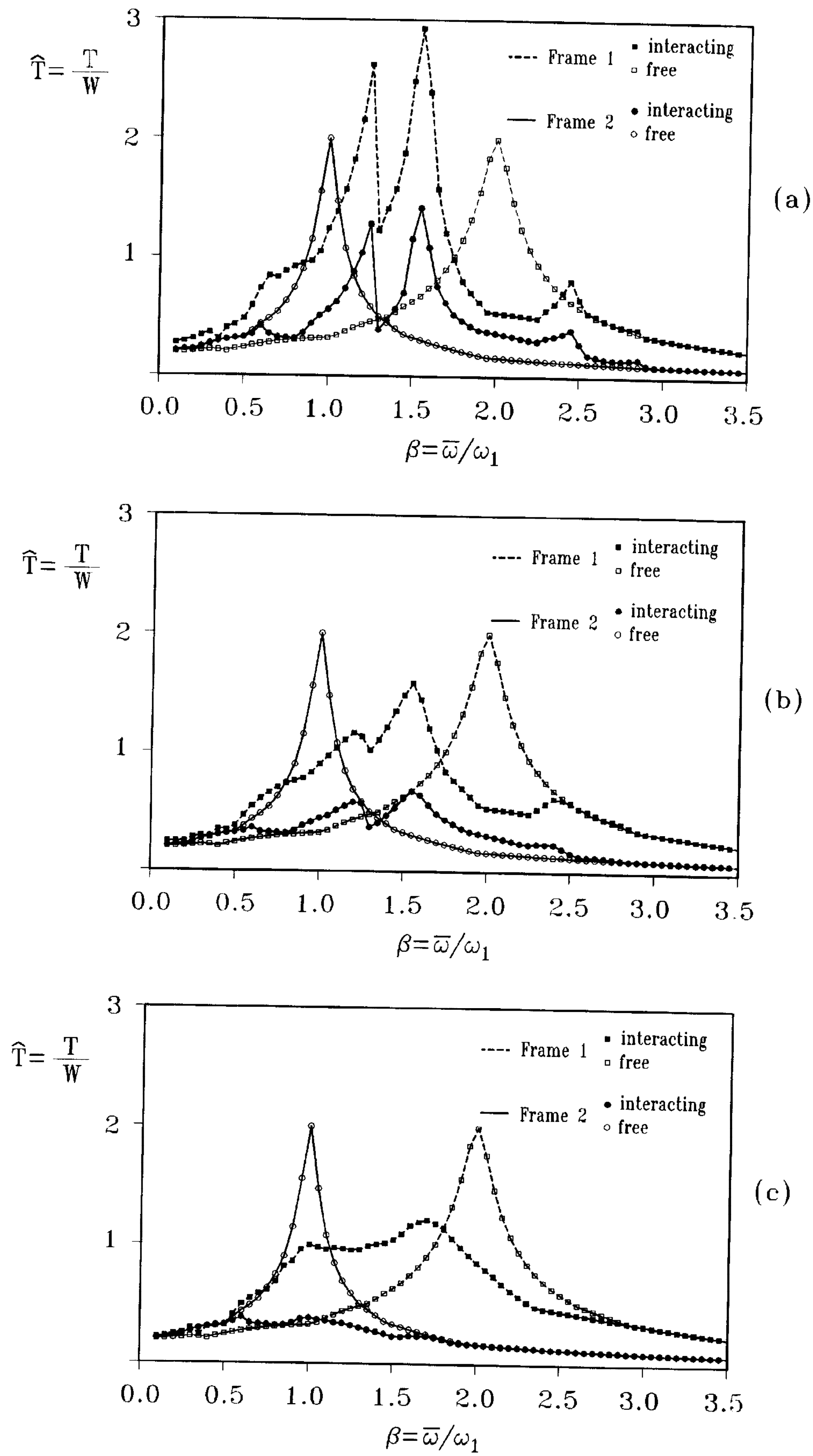

Fig. 2. Amplitude of the frequency response functions of the shear force at the base of two single storey frames: $\lambda^2 = 200\ s^{-1}$, $\alpha = 0.25$, $\beta = 1$, $\gamma = 0.5$, $\bar{\xi} = 0.05$, $\bar{\delta} = 0.25$. a) elastic impacts: $e = 1.00$, $\eta = 0$; $\mu = \bar{\varepsilon} = 0$; b) partially elastic impacts: $e = 0.75$, c) gap elements: $e = 0.75$, $\eta = 0.125$, $\mu = 1$, $\bar{\varepsilon} = 0.5$.

Furthermore in the range around the resonance of the independent stiffer frame the curve of the more flexible frame shows an increase, while the curve of the stiffer one shows a reduction. On the contrary in the range around the more flexible frame resonance the opposite situation occurs. The same remarks hold for the graphs of Fig. 2b, which refer to partially elastic impacts for e = 0.75, but the frequency response functions of the interacting frames present obviously a reduced amplitude. The graphs of Fig. 2c refer to unilateral gap elements interposed between the two frames and e = 0.75. This case appears to be the more favourable. In fact the frequency response functions present reduced amplitudes and the more flexible frame is nearly always less stressed.

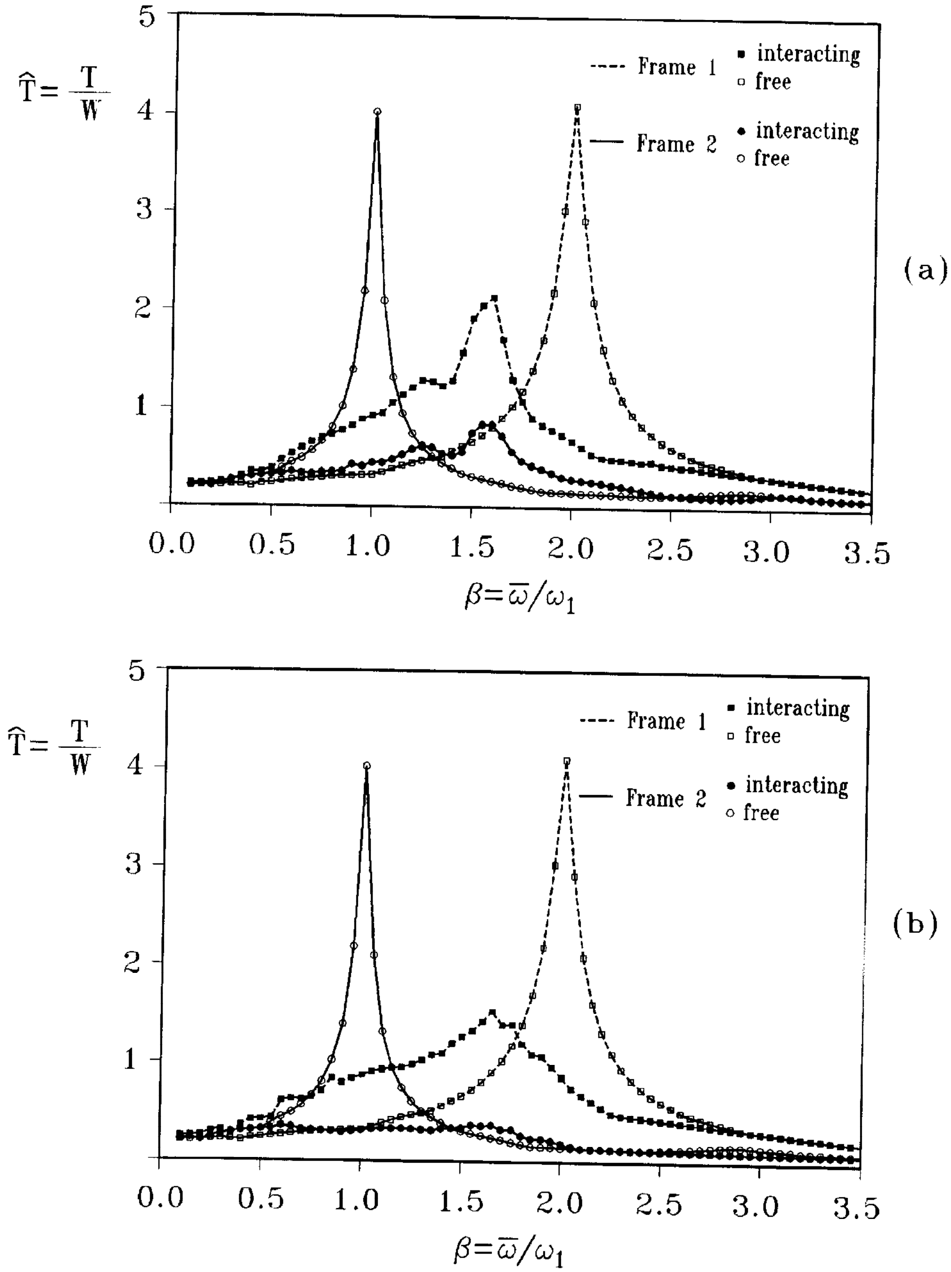

Fig. 3. Amplitude of the frequency response functions of the shear force at the base of two frames having three storeys: $\lambda^2 = 400\ s^{-1}$, $\alpha = 0.25$, $\beta = 1$, $\gamma = 0.5$, $\bar{\xi} = 0.05$, $\bar{\delta} = 0.5$. a) partially elastic impacts: $e = 0.75$, $\eta = \mu = \bar{\varepsilon} = 0$; b) gap elements: $e = 0.75$, $\eta = 0.125$, $\mu = 2$, $\bar{\varepsilon} = 1$.

The second structural system considered is composed of two frames, each having three storeys. The results in terms of frequency response functions of the dimensionless shear force at the base are reported in Fig. 3a and in Fig. 3b, which refer to the cases of partially elastic impacts and unilateral gap elements respectively. The frequency range including only the first natural frequencies of the two independent frames is displayed. The trend of the curves is practically similar to that of the single storey frames and the same comments continue to be valid.

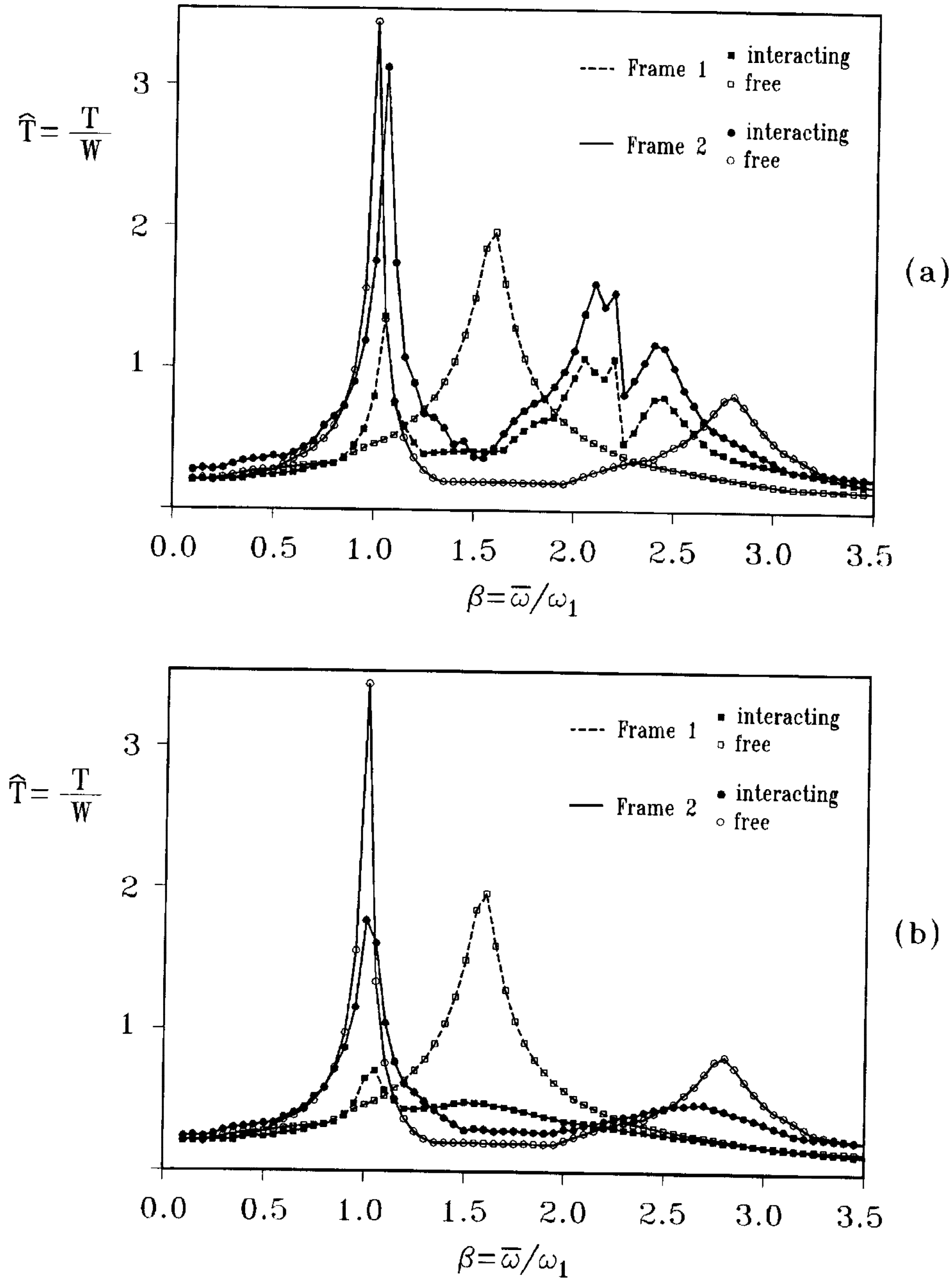

Fig. 4. Amplitude of the frequency response functions of the shear force at the base of two frames having one and three storeys: $\lambda^2 = 200\ s^{-1}$, $\alpha = 2$, $\beta = 1$, $\gamma = 1$, $\bar{\xi} = 0.05$, $\bar{\delta} = 0$. a) elastic impacts: $e = 1.00$, $\eta = \mu = \bar{\varepsilon} = 0$; b) gap elements: $e = 1.00$, $\eta = 0.5$, $\mu = 1$, $\bar{\varepsilon} = 0.5$.

The third system taken into account is composed of two frames having one and three storeys respectively. The same cases of the previous system have been considered and the results are reported in Fig. 4a and in Fig. 4b. The displayed frequency range includes the first two natural frequencies of the three storey frame and the fundamental frequency of the single storey one. As it may be noticed the curves of the interacting frames are more involved than those of the previous cases. When no gap elements are interposed, Fig. 4a, the response amplitude is dramatically increased for both frames in the frequency range between the fundamental frequency of frame 1 and the second natural frequency of frame 2. Furthemore frame 1 is more stressed close to the first resonance of frame 2. However it should be noticed that, in this case, the interposition of unilateral gap elements leads to a considerable decrease in the response amplitude, as it may be seen in Fig. 4b.

CONCLUSIONS

A numerical method for the evaluation of the dynamical response of two colliding adjacent buildings has been presented. A simplified but physically significant structural system, which allows to elucidate the main aspects of pounding, has been considered. The case of elastic or partially elastic impacts as well as that of interposed unilateral gap elements have been examined in order to investigate when this phenomenon can be exploited to control the response amplitude. The parameters which govern the behaviour of the system have been pointed out and some numerical applications in the frequency domain have been developed with reference to a few but significant sample structures. The results show that pounding strongly modifies the dynamical characteristics of the colliding structures, moving their natural frequencies and varying the peak amplitudes of their frequency response functions. Whether pounding may be beneficial or not to reduce the response amplitude essentially depends on the frequency content of the excitation and on the properties of the interposed unilateral gap elements. Further investigations are needed in order to examine the response amplitude sensitivity to a variation of the structural parameters. This will be the subject of a paper to come.

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