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Seismic interaction between adjacent buildings under second-order geometric effects

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ABSTRACT: The paper deals with a numerical approach for the problem of earthquake interaction among neighboring buildings when unilateral elastoplastic/ elastic contact under second-order geometric and other instabilizing effects can take place. The method is based on formulating the problem by the finite element method as an inequality one and on solving this by the average-acceleration method of time-discretization and nonlinear mathematical programming. Some results concerning a two-building system under P-Delta effects are given in a numerical example.

1 INTRODUCTION

Earthquake induced pounding between adjacent buildings has been recognized -see e.g. Newmark and Rosenblueth (1971)- as one of the main usual causes of significant damages in seismically active regions. This holds especially for market-areas of cities, where the building codes, due to various socioeconomic reasons, allow partial or full contact between neighboring buildings (Bertero (1987)).

From mathematical point of view, the governing conditions of the relevant problem are equalities as well as inequalities. The latter ones concern on the one hand the possibility to be appeared compressive stresses only (no tension) on the interface, and on the other hand the appearence of retirement relative displacements (no penetration) for the same interface points where unilateral contact can take place. So, the problem belongs to so-called inequality problems of mechanics, for which a mathematical study can be done by the variational inequality concept - see Panagiotopoulos (1985).

As regards numerical results, some interesting studies concerning simplified models of single-degree-of-freedom systems have been reported by Wolf and Skrikerud (1980), Anagostopoulos (1988), Penelis and Athanassiadou (1989). These investigations are based on a parametric trial-and-error approach. A more realistic numerical treatment of such inequality problems in earthquake engineering for multidegree-of-freedom structural systems has been already presented by the first author (A.A.L.) in a series of papers, see e.g. Liolios (1984,1988,1989,1990,1991).

The purpose of this paper is to deal with a numerical approach for the above outlined dynamic inequality problem when some instabilizing effects are taken into account. These effects concern here elastoplastic-softening/fracturing behaviour for unilateral contact and P-Delta effects. The method is based on a double discretization, in space by the finite element method and in time by the average acceleration method, and on solving a non-convex linear complementarity problem in each time-step. The proposed method is applied, finally, to a civil engineering example and some conclusions are discussed.

2 METHOD OF ANALYSIS

A system of two only adjacent buildings (A) and (B) is considered here for simplicity. Certainly, the extension to systems with more than two buildings is straightforward.

First, following Liolios (1984), the system is discretized in space by the finite element method. Any two associated nodes i_A and i_B on the interface are considered as connected by a unilateral constraint, normal to the interface. The stress r_i, positive when it is compressive, and the corresponding shortening relative displacement

v_i of the i-th unilateral constraint satisfy the following, in general non-convex, constitutive relation:

$$r_i \in \partial R_i(v_i, g_i). \qquad (1)$$

Here $g_i(t)$ is the existing gap at time t between nodes i_A and i_B, ∂ is Clarke's generalized gradient and R_i is the symbol of non-convex superpotential - see Panagiotopoulos (1985). Relation (1) expresses in a general mathematical way the unilateral frictionless elastoplastic contact taken into account hardening/softening, unloading/reloading, fracturing etc. behaviour. For simplicity, the case of frictionless contact is studied here. The frictional case, which is more complicated, can be investigated in a way similar to that of Liolios (1989, 1991). As known, softening/fracturing behaviour corresponds to descending branches in the diagram of (1), and usually has instabilizing effects to the numerical procedures and the structural response. Moreover, the elastoplastic behaviour of unilateral constraint permits us to assume that local impact phaenomena have no significant influence to the global building response.

Now, for the numerical treatment of the problem, the rel. (1) is piece-wise linearized in a way similar to that used by Maier (1971,1973) in elasto-plasticity. So, introducing the nonnegative multipliers w_i, rel. (1) is equivalent to the following linear complementarity conditions:

$$r_i = p_i(v_i - g_i + w_i) + c_i \dot{w}_i, \qquad (2a)$$

$$w_i \geq 0, \qquad r_i \geq 0, \qquad (2b,c)$$

$$r_i w_i = 0. \qquad (2d)$$

Here c_i is the damping coefficient and p_i the stress function for the i-th unilateral constraint. Dots over symbols denote, as usually, time-derivatives.

Further, the incremental global equations of dynamic equilibrium for the two buildings (A) and (B) due to a seismic ground displacement history $\underline{x}_g(t)$ are written in matrix notation:

$$\underline{M}_A \Delta\ddot{\underline{u}}_A + \underline{C}_A \Delta\dot{\underline{u}}_A + (\underline{K}_A + \underline{G}_A)\Delta\underline{u}_A = = -\underline{M}_A \ddot{\underline{x}}_g + \Delta\underline{r}, \qquad (3a)$$

$$\underline{M}_B \Delta\ddot{\underline{u}}_B + \underline{C}_B \Delta\dot{\underline{u}}_B + (\underline{K}_B + \underline{G}_B)\Delta\underline{u}_B = = -\underline{M}_B \Delta\ddot{\underline{x}}_g - \Delta\underline{r}. \qquad (3b)$$

Here, as usually, $\underline{M}$, $\underline{C}$ and $\underline{K}$ denote the mass, damping and current (tangent) first-order (linear elastic) stiffness matrices, respectively. $\underline{G}$ is the symmetric constant geometric stiffness matrix, depending linearly on preexisting constant stresses. Thus, via the term $\underline{G}\underline{u}$ alone the geometry changes affect the equilibrium (second-order geometric effects) -see e.g. Maier (1971), Corradi and De Donato (1975), Chen and Lui (1987). $\underline{u}(t)$ is the node-displacement vector (relative to ground); Δ denotes increment; and finally, $\underline{r}$ is the vector of interaction forces between (A) and (B) with elements satisfying rels. (1)-(2).

Thus the problem consists in computing the time-dependent set $\underline{u}_A$, $\underline{u}_B$, $\underline{r}$, $\underline{w}$ and $\underline{g}$ satisfying (1)-(3) for given initial conditions and $\underline{x}_g(t)$.

Due to inequality conditions, the problem is a nonlinear one, even in the case of linear structures. To discretize this problem in time, use is made of the average-acceleration method, which belongs to Newmark's family of step-by-step direct time integration methods -see e.g. Weaver and Johnston (1987). So we substitute in (3) for every time-step

$$\Delta\ddot{\underline{u}} = c_1 \Delta\underline{u} + \underline{a}, \qquad (4a)$$

$$\Delta\dot{\underline{u}} = c_2 \Delta\underline{u} + \underline{b}, \qquad (4b)$$

where $\underline{a}$, $\underline{b}$ known quantities from previous time-steps and

$$c_1 = 4/(\Delta t^2), \quad c_2 = 2/\Delta t \qquad (5)$$

are method parameters. After the above manipulation we arrive eventually to a linear complementarity problem -see also Liolios (1988,1989)- of the form

$$\underline{z} \geq \underline{0}, \qquad \underline{D}\underline{z} + \underline{d} \leq \underline{0}, \qquad (6a,b)$$

$$\underline{z}^T \cdot (\underline{D}\underline{z} + \underline{d}) = 0. \qquad (6c)$$

This problem is solved by known algorithmes of nonlinear optimization - see e.g. Panagiotopoulos (1985) or Maier (1973). Thus, in each time-step Δt is computed which unilateral constraints are active and which are not.

The so-obtained results concern the response of the coupled system, where

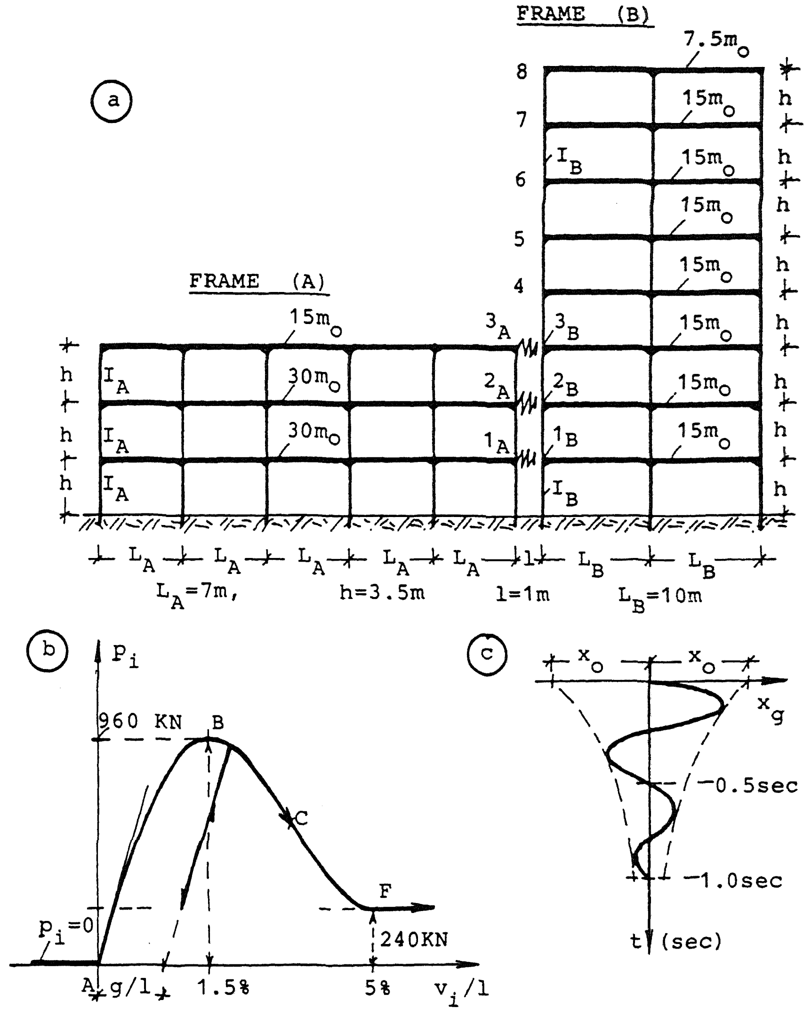

Fig. 1. Numerical example

the unilateral contact and the second-order geometric effects are taken into account. To compare these results with corresponding ones for the uncoupled system, where the interaction effects are not taken into account and the structures are designed as beeing entirely independent (as was usual until recently in most aseismic computations), the following influence coefficients are introduced:

$$\lambda_i = (Q_i^c - Q_i^u).100/ Q_i^u . \qquad (7)$$

Here Q^u and Q^c are the absolutely maximum values, which a response quantity Q takes during the seismic excitation when the structures are uncoupled and coupled, respectively.

3 NUMERICAL EXAMPLE

In the building system of Fig. 1a, the frame (A) is of reinforced concrete with elastic modulus E_b =3.4*10^{7}KN/m^2 and column sections 40/60 in cm, and the frame (B) is of steel with E_s = 21*10^{7}KN/m^2 and columns IPBl 500. Damping ratio is 5% for (A) and 3% for (B). Both frames are considered as having rigid beams with total vertical loads $a_i m_o g$, where $m_o g$ =98.1 KN, g = 9.81 m/sec^2 and a_i=coefficients as in

Fig. 1a. Unilateral contact can take place at joint-points i_A, i_B, where i=1,2,3. The corresponding to (2a) function p_i is assumed to be as shown in Fig. 1b, where AB and BCF are parabolas of 2-nd and 3-rd degree, respectively. The above simulation of unilateral contact is certainly a very complicated task and can be estimated on the basis of experimental results. P-Delta effects for the steel frame (B) are taken into account. The plane system is subjected to an horizontal earthquake ground displacement

$$x_g(t) = x_o e^{-2t} \sin(4\pi t) \qquad (8)$$

with x_o = 10mm and diagram as in Fig. 1c. The herein presented numerical approach has been applied to estimate quantitatively the interaction effects on the seismic response of frames (A) and (B).

From the so-obtained results are shown indicatively in Table 1 only those concerning the storey-shear-forces of the frames (A) and (B).

Table 1. Influence coefficients (in %) for the storey shear forces

Storey	Frame (A)	Frame (B)
1	-24.19	+67.42
2	-15.98	+12.54
3	-25.67	+26.41
4		+119.73
5		+46.88
6		+46.05
7		+41.79
8		+44.96

As the table results show, the uncoupled stress-state of the three-storey frame (A) is reduced about 16% - 26% due to interaction. On the contrary, the uncoupled stress-state of the eight-storey frame (B) is increased about 13% - 120%. As was expected, the most significant increase is for the 4-th floor of (B). Thus, if the columns of this floor are designed without taken into account the seismic interaction effects, then these columns are overstressed about 120% more than the designed capacity. This remarkable result shows the significance of computing the interaction influence on the seismic response of adjacent buildings.

4 CONCLUSIONS

The herein presented numerical method can be used effectively in practical civil engineering applications, where a quantitative estimation of the seismic interaction between adjacent buildings under second-order effects is required. For this purpose, the realization on computer of the method seems to be indispensable. This is obtained by using available computer codes of the finite element method, the direct time integration methods and nonlinear optimization.

As the results of the numerical example show, the interaction influence on the earthquake response of neighboring buildings may be significant. Therefore the usual aseismic design and control without taking into account such a possible interaction under second-order effects may be no realistic. Certainly, the sufficient aseismic joint among adjacent buildings seems to be an effective rule for seismically active regions. If this rule can be applicable, and if the simulation of the unilateral contact behaviour can be done in a realistic way (e.g. by experimental results), then the seismic joint gap can be adjusted suitably by the herein procedure. So, a parametric application of the presented method, having as one parameter the joint gap, can be used effectively to control the seismic interaction effects in a desirable level.

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