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Earthquake induced pounding in adjacent buildings

K.V. Spiliopoulos
National Technical University of Athens, Greece

S.A. Anagnostopoulos
University of Patras, Greece

ABSTRACT: The response of adjacent buildings in a row, subjected to strong earthquake motions is analyzed, taking into account their mutual pounding resulting from insufficient separation distances. Each building is idealized as a series of lumped mass, shear beam type MDOF system with bilinear force-deformation characteristics. One translational degree of freedom is allowed for every mass, except for the foundation mass which, in addition, can rotate to permit rocking motion. It is assumed that pounding can occur only at the floor levels, where the masses are lumped. Impacts are simulated by means of viscoelastic elements. Amplification of the response due to pounding is found to depend primarily on the period ratios, mass ratios and different heights of the adjacent buildings. Suggestions are given for possible introduction into codes of certain conditions, as an alternative to the seismic separation requirement.

1 INTRODUCTION

Studies from recent earthquakes that have struck major cities, have shown that pounding between adjacent buildings can cause varying degrees of structural, e.g. Bertero 1987 and non-structural damage, e.g EERI 1989. Although many modern codes may include separation requirements so that pounding could be avoided, large sections of cities have been built before any such requirements were introduced. The separation requirement, even for new buildings, has considerable architectural implications on small urban lots, because to make provision for the worst-case condition could result in large building separations and significant loss of usable space. On the other hand, the idea of urban buildings with small spaces between them, creates a very difficult maintenance problem.

The problem of earthquake induced structural pounding has been studied primarily for pairs of buildings, e.g. Wolf and Skrikerund 1980, Liolios 1988, Papadrakakis, Mouzakis, Plevris, Bitzarakis 1991. All the aforementioned studies indicate that pounding can amplify or reduce the response of pairs of adjacent structures.

When there are more than two buildings in a row, which is a common case in city blocks, the problem of pounding appears quite different since the interior buildings are subjected to two-sided impacts, whereas the end ones to one-sided impacts (Anagnostopoulos 1988, Anagnostopoulos and Spiliopoulos 1992).

The present paper addresses the problem of earthquake induced pounding in rows of neighbouring buildings using nonlinear, MDOF models on viscoelastic foundation. Results are presented in the form of mean amplification ratios of story shears (elastic response) and ductility factors (inelastic response) for five earthquake motions using as a reference the response without pounding.

For a more detailed discussion one is referred to Anagnostopoulos and Spiliopoulos 1992.

2 IDEALIZATION-ASSUMPTIONS

In the present work, each of a number of adjacent buildings is idealized as a MDOF, close-coupled (shear beam type) system with bilinear interstory resistance characteristics and masses lumped at the floor levels (Fig.1). Rayleigh type modal damping 5% of critical in the first two modes of each structure is specified. Foundation flexibility is accounted for by means of appropriate rocking and translational spring-dashpot elements. It is further assumed that floor elevations are the same for all buildings and pounding is simulated with viscoelastic impact elements, each of them consisting of a spring with constant s_j and a dashpot with constant c_j. These elements become active only when the corresponding neighbouring floor masses come into contact. One translational degree of freedom is allowed for every mass, except for the mass of the foundation which in addition, can rotate to permit rocking motion. All systems are subjected to the same ground acceleration $u_g(t)$, which implies that any effects of phase difference due to travelling waves are neglected.

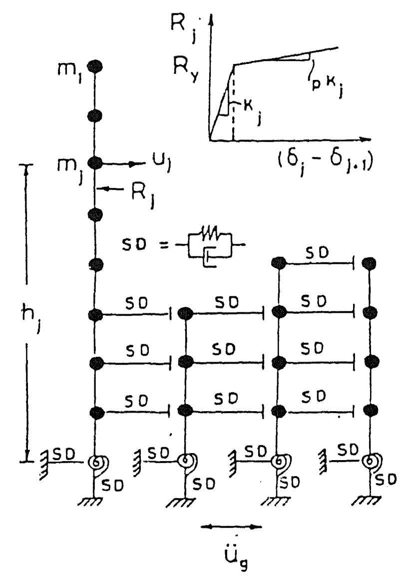

Figure 1. Idealization of adjacent buildings.

3 EQUATIONS OF MOTION

Let u_j be the horizontal displacement of mass m_j of one of the systems relative to the ground, u_b the translation and ϕ_b the rotation of the foundation, δ_j the part of u_j due to structural deformations. Then one can write u_j as:

$$u_j = \delta_j + u_b + h_j\, \phi_b \tag{1}$$

where h_j is the elevation of mass m_j from the base (Fig.1). Two alternative formulations of the equations of motion are possible: either in terms of the total displacements u_j, which include the rigid body component $u_b+h_j\phi_b$ or in terms of the structural deformations δ_j. The first formulation results in a diagonal mass matrix, but in a non-symmetric damping matrix due to impact elements. In the second formulation the opposite happens: the resulting mass matrix is non-diagonal, whereas the damping matrix due to impact elements is symmetric. For computational efficiency the second formulation has been selected. Thus the equation of motion for mass m_j can be written as follows:

$$m_j(\ddot{\delta}_j+\ddot{u}_b+h_j\ddot{\phi}_b)+\sum_{i=1}^{n} c_{ji}\,\dot{\delta}_i +F_j+R_j= -m_j\ddot{u}_g \tag{2}$$

where dots indicate derivatives with respect to time, c_{ji} = damping coefficients (Rayleigh type), F_j = impact force which acts only when contact occurs, R_j = restoring force due to structural resistance, n= number of floors of the system considered, $\ddot{u}_g$ = ground acceleration. Since impact force starts acting when contact occurs from either side of the system considered, we keep track of the instantaneous distances between mass m_j and its neighbouring two masses (Figure 2):

$$v_l = u_{j(l)} - u_j - d_{(l)}$$
$$v_r = u_j - u_{j(r)} - d_{(r)} \tag{3}$$

where $u_{j(l)}$, $u_{j(r)}$ = total displacements of the two masses adjacent to mass m_j on the left and right side, respectively, and $d_{(l)}$ and $d_{(r)}$ the separation distances when the systems are at rest. Thus, the conditions for contact between mass m_j and its neifhbouring masses are $v_l>0$ and $v_r>0$, for left and right contact, respectively. When concurrent left and right impacts occur, impact force F_j will be given by the equation:

$$F_j=F_{jr} -F_{jl} =(s_r v_r +c_r\dot{v}_r)-(s_l v_l +c_l\dot{v}_l) \tag{4}$$

It is obvious that the first or the second parenthesis will be zero if no right or left impacts occur; s_r, c_r , are the right and s_l , c_l are the left spring and damping constants of the impact elements.

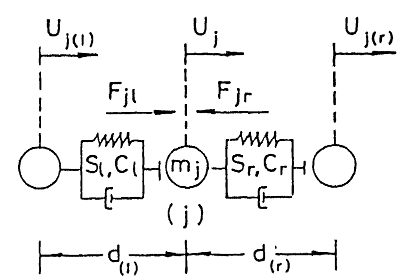

Figure 2. Notation for impact forces on mass m_j.

The nonlinear restoring forces R_j are computed numerically from the bilinear force-deformation relations of each story (Fig.1) :

$$R_j = k_j^t(\delta_j - \delta_{j+1}) - k_{j-1}^t(\delta_{j-1} - \delta_j) \tag{5}$$

where k_j^t = tangent stiffness of story j.

Equation (2) can be written for all the n masses of each system. Since two additional degrees of freedom for the system exist, two more equations expressing the dynamic equilibrium of all the horizontal forces and their moments about the base are written. The first of these two equations involves the mass m_b and the second one, the moment of inertia I_b of the foundation.

If N systems in a row are present, the total number of degrees of freedom for the whole configuration is: $\sum_{i=1}^{N} n_i + 2N$ where n_i is the ith system in the row.

Coupling of the equations of two adjacent systems exists only when one or more masses come into contact. In matrix form, one can write the equations for the whole configuration :

$$[M]\{\ddot{U}\} + [C]\{\dot{U}\} + [S]\{U\} + \{R\} = -\ddot{u}_g\{m\} \quad (6)$$

where [M]=mass matrix, {U}=displacement vector of all the unknown d.o.f., $[C]=[C]_R + [C]_I$ is the total damping matrix, $[C]_R = \alpha[M] + \beta[K]$ = structural damping matrix of Rayleigh type, [K]=elastic stiffness matrix, {R}=vector of structural resistances, {m}=right-hand side mass vector, $[C]_I$ and [S] = damping and stiffness matrix due to dashpot and spring constants, respectively, of active impact elements. [M] and [K] are block diagonal matrices. Each one consists of submatrices of the individual systems, which are of the usual form of a close-coupled shear beam-type model.

The damping matrix $[C]_I$ is formed by submatrices 6x6, each of them corresponding to an impact element currently active. Each submatrix couples the degrees of freedom at the two ends of the impact element, as well as the degrees of freedom associated with the foundation masses of the two colliding buildings. If c is the dashpot constant of the impact element between masses m_i and m_j and h is the elevation of the two masses from the base, the 6x6 submatrix $[C]_{ij}$ is given by:

$$[C]_{ij} = \begin{bmatrix} c & c & ch & -c & -c & -ch \\ & c & ch & -c & -c & -ch \\ & & ch^2 & -ch & -ch & -ch^2 \\ & & & c & c & ch \\ & \text{SYM} & & & c & ch \\ & & & & & ch^2 \end{bmatrix}$$

The first and 4th columns of $[C]_{ij}$ correspond to the translation of masses m_i and m_j respectively, the 2nd and 5th columns correspond to the base translations of the two buildings and the 3rd and 6th columns correspond to the base rotations.

The product [S]{U} as well as the resistance vector {R} are formed directly in the right hand side of the algebraic system of equations to which the differential equations are transformed to be solved numerically.

Finally, there is no need to assemble the [K] matrix of the configuration, since only the individual stiffness matrices of the systems are evaluated for an initial eigensolution.

4 COMPUTATIONAL CONSIDERATIONS

The equations of motion are solved numerically using central differences with Newmark's method to start the solution. Since the impact element stiffnesses are generally large compared to the story stiffnesses, the time step required to adequately reproduce the impact forces is quite small. Therefore it becomes necessary for computational efficiency to use two different time steps: a large time step, applied when no impact takes place and a finer time step which is applied when two or more masses come into contact. As a rule, the large time step should be less than about 1/10 the lowest period of all natural periods of the buildings in the group and the finer time step should be less than about 1/10 the lowest local period, determined from the impact element stiffnesses and the associated masses. For computational efficiency also, matrices are stored in skyline form, whereas a book-keeping approach of the current active impact elements is applied.

5 PARAMETRIC STUDIES

Parametric investigations were performed on a group of 5-story and a group of 10-story systems. Four 5-story systems with fundamental periods of 0.27sec, 0.36sec, 0.48sec and 0.60sec as well as two 10-story systems with fundamental periods of 0.78sec, 1.03sec were used. Each system in the group has the same mass in a given floor, whereas the stiffnesses of each system, assumed to vary lineraly with height, were selected to produce the desired periods. Yield levels for the inelastic solutions were taken equal to the story shears which were determined in accordance with the UBC code (1988). Foundation constants were determined using spread footings and stiff soil conditions. A sensitivity study has shown that the response of the pounding systems is not very sensitive to changes in the impact elements' stiffnesses or damping constants. The stiffnesses of the impact springs were chosen so that the local periods of the mass - impact spring are below the lowest translational periods of the pounding systems. The damping constants of the impact elements were estimated as suggested by Anagnostopoulos 1988, for a coefficient of restitution r=0.50. Elastic and inelastic analyses have been performed for 5 earthquake records listed in Table 1. The scale in the last column of the Table was computed by equating the Arias intensities of these motions to the Arias intensity of the El Centro record. For elastic analyses results are presented in terms of mean values of ratios (V/V_o) and for inelastic analyses in terms of mean values of ratios (μ/μ_o), for the 5 motions. V and μ are the maximum story shear and ductility factor of the pounding building respectively, whereas V_o and μ_o are the corresponding maximum story shear and ductility factor of the same building without pounding.

5.1 System configuration

The great many ways in which different buildings can be arranged in a row, constitute one of the major difficulties in studying the problem at hand. Four different configurations concerning the two buildings with periods T=0.36 and T=0.60 sec are indicated as an inset at the top of Figure 3.

Table 1. Earthquake motions used in analyses

Record	u_{gmax} (g)	Duration (sec)	Scale
El Centro (1940)- NS	0.35	10	1.00
Taft (1952)-S69E	0.18	15	1.75
Eureka (1954)-N79E	0.26	10	1.33
Olympia (1949)-N86E	0.28	23	1.25
Parkfield (1966)-65E (Array No.2)	0.49	10	0.82

The response of the system with period T=0.36 sec is plotted in the two graphs at the bottom of the figure. The first two configurations are considered together and in the line corresponding to the 2-system case the maximum of the mean values of ratios from the five earthquake motions are plotted, whereas for the 3 and 4 system configurations, the average values of the mean ratios of the two end buildings are plotted.

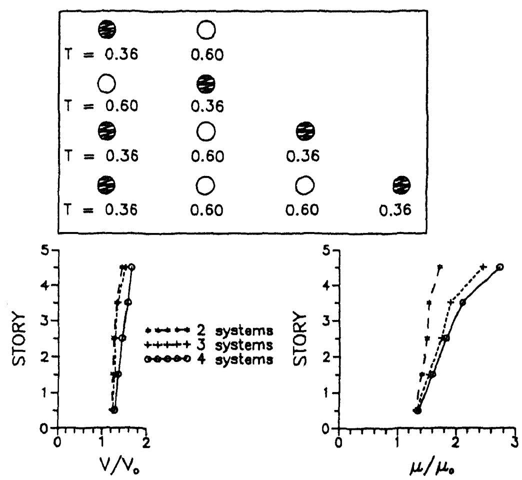

Figure 3. Effects of pounding on elastic and inelastic response of 5-story buildings in different configurations.

It can be seen that there are not significant differences between the results of the three cases and especially between the 3 and 4 system cases. A conclusion, therefore, can be drawn that the effects of multiple collisions on the response of any building in a given configuration are predominantly determined by the properties of the adjacent buildings. The collisions of buildings that are not adjacent to the building considered and thus do not interact with it directly, do not influence this building's response appreciably. Therefore, a 3- system configuration, in which one and two-sided impacts can take place, is sufficient to study the problem of system configuration.

Figure 4 shows the effects of one-sided or two-sided pounding on the response of the 5-story system with T=0.36 sec. The three lines in each graph correspond to the configurations shown at the top of the Figure, each of them characterized by the ratio p of the period of the system examined (in this case T=0.36) to the period of the adjacent system. Results from elastic solutions are given in terms of shear ratios V/V_o (upper graphs) and from inelastic solutions in terms of ductility ratios μ/μ_o (lower graphs). If the system examined is between the other two (two-sided impact), then these ratios are the mean values for 5 earthquake motions, whereas if the system is at the two ends (one-sided impact), then the larger of the mean ratios from the two end systems are plotted.

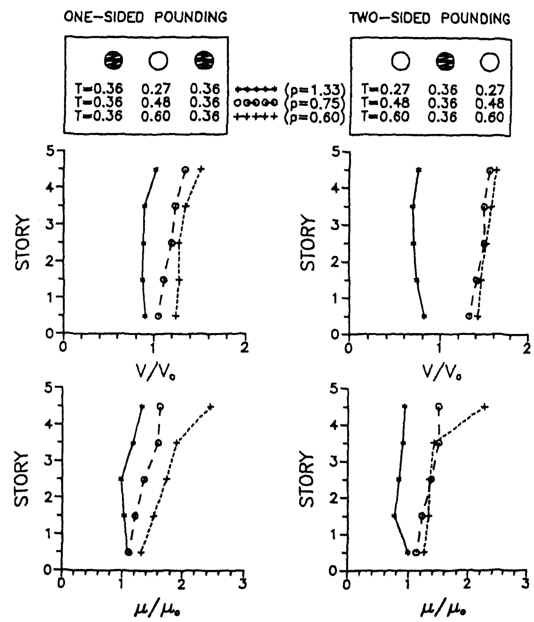

Figure 4. Effects of pounding on elastic and inelastic response of a 5-story system.

It is observed that when the adjacent system is more flexible ($p<1$), pounding response is amplified. As the value of p becomes lower, the amplification increases. Moreover, there is no much difference in the amplification of the response if we compare the results from one-sided and two-sided cases except for the inelastic case with p=0.60, where for the one-sided impacts we get larger amplifications. For the case of $p>1$ the elastic response due to pounding is reduced. The reduction is greater in the case of two-sided pounding as would be expected since the system pounds against stiffer ones on both sides. On the other hand, the inelastic response may be amplified for the case of one-sided pounding, as can be seen in the graph of Fig.4

Next the effects of soil conditions on the response due to pounding are investigated. Since effective system periods would increase as a result of a softer soil, the consequences of pounding are expected to be reduced. This was certified by analyzing a 10-story system configuration with foundation constants corresponding to a soil 5 times softer than the original. The periods of the systems shown at the top of Figure 5 were increased from 0.78 sec and 1.03 sec to 0.92 sec and 1.14 sec, respectively. The maximium reduction that can be observed in the mean μ/μ_o ratios of T=1.03 sec goes up to about 30%.

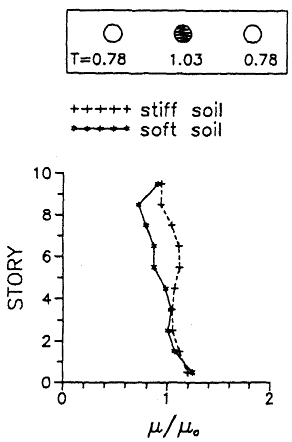

Figure 5. Effects of soil conditions on the inelastic seismic response of a 10-story system subjected to pounding

5.2 Buildings of unequal heights

The effect of pounding for buildings of unequal heights is examined next. The 10-story system with T=1.03 sec and the 5-story system with T=0.36 sec, placed at a distance of 5mm were analyzed for the 5 earthquake motions. Results are presented in Fig 6a in terms of mean and maximum ductility ratios.It can be observed that pounding causes large ductility demands on the low building. On the other hand, the effects on the response of the tall building are not so significant.

Much more serious consequences due to pounding appear (as shown in Fig 6b) for the tall building when the low building is stiff and massive and thus restrains the motion of the lower half of the tall building.

5.3 Seismic separation

As the separation distance increases the effects due to pounding decrease. In Figure 7 results from testing seismic separation requirements of UBC 1988, as well as Eurocode No.8, 1989 are presented. Two

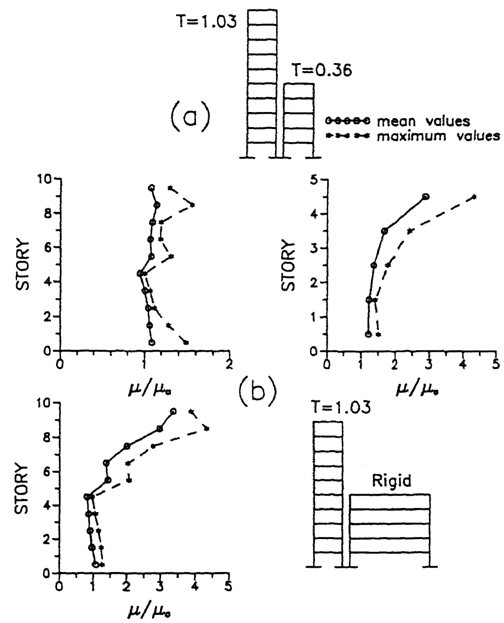

Figure 6. Effects of pounding on the inelastic response of two buildings with different heights.

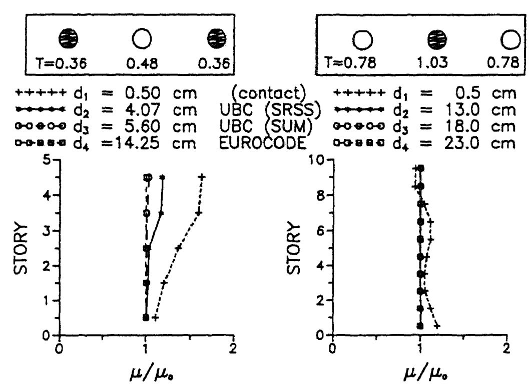

Figure 7. Effects of separation distance on the inelastic seismic response of 5 and 10-story systems subjected to pounding.

configurations were analyzed one with 5-story systems and one with 10-story systems. Four distances were examined: d_1=0.5cm (practically in contact), $d_2 = \sqrt{\delta_1^2 + \delta_2^2}$, where δ_1 and δ_2 are the UBC design displacements of the colliding buildings, $d_3 = d_1 + d_2$ = the UBC required separation distance and d_4 = the separation distance of Eurocode No.8.

Although the model is highly idealized and therefore not very appropriate to check code provisions, one can see that the UBC specified separation is sufficient to preclude pounding for the 10-story systems but not quite so for the 5-story systems (although in this case the effects of collisions are almost negligible). The more conservative Eurocode 8 separation requirement is adequate for both the 10-story and 5-story systems.

5.4 Effects of relative mass size

The effect of different relative mass sizes on the amplification response of the pounding buildings was examined next. Three 5-story systems were used as indicated at the top of Figure 8. The larger of the mean ductility ratios of the two end systems were plotted for the five earthquakes and for four mass sizes of the system in the middle.

In all cases, properties of the end systems were unchanged, while the stiffness and yield levels of the middle system was changed in proportion to the mass, so that the periods and yield displacements be kept constant, same as in the basic design, in which the three systems have equal masses. It can be seen that when the masses of the middle building are reduced 5 times, there is practically no amplification of the response due to pounding, whereas more than 50 % amplification appears when the masses of the middle building increase 5 times.

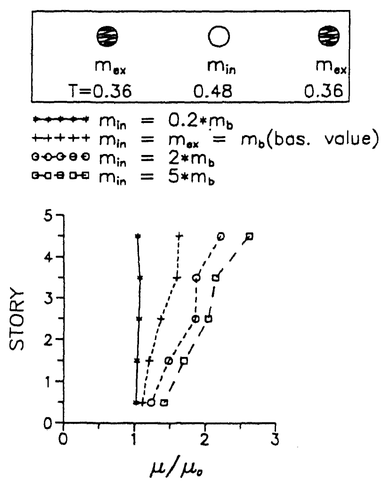

Figure 8. Effects of mass size on the inelastic seismic response of 5-story systems subjected to pounding.

6 CONCLUSIONS

The problem of earthquake induced pounding of several buildings in a row has been investigated, using lumped mass, MDOF, shear beam type idealisation. Subject to the limitations of the model used, the following conclusions may be drawn:

1. Amplification of the response, due to pounding, of a building depends mainly on its period and mass in relation to the periods and masses of its adjacent buildings. As a rule, when the masses are similar, the response of the stiffer building is amplified when it pounds against a more flexible one. When there is large difference in the masses, the building with the smaller mass is highly penalised.

2. For adjacent buildings of unequal heights, serious problems can be caused from pounding. Due to the differences in their masses and periods, the small building is highly overstressed. On the other hand, when the lower building is stiff and massive, the upper part of the taller building is greatly penalised.

3. From the examples that are considered here, the seismic separation gaps introduced by the codes seem to be generally adequate to prevent pounding or highly reduce its effects.

4. Differences in heights, periods and masses of adjacent buildings seem to be the most crucial factors that affect the response of pounding buildings. It may, therefore, be possible to introduce into the codes conditions and provisions as an alternative to the seismic separation requirement.

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