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DESIGN CRITERIA FOR BUILDINGS SUBJECTED TO POUNDING

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ABSTRACT

During various recent earthquakes, namely Mexico 1985, Northridge 1994 and Kobe 1995, pounding between adjacent buildings has induced their total or partial structural collapse. The analysis of seismic design criteria for buildings that may be affected by pounding from adjacent constructions and the establishment of rules that diminish the potential negative effects of such phenomenon are thus topics of relevant interest.

In this study, to analyse rules that can be applied in general terms, different geometrical configurations of adjacent buildings are considered as well as the influence of several parameters such as the seismic motion itself; the relative height of adjacent buildings, which influences not only their dynamic characteristics but also defines the zones more prone for impact; the relative location of the buildings when situated in a row and the gap size between adjacent buildings.

The consequences of pounding are analysed in terms of the comparison of relevant structural response parameters evaluated with and without the consideration of structural impact.

The main conclusions of this study are that there should be a minimum gap between adjacent buildings to avoid the negative effects of impact and that, when this objective can not be reached, special care should be taken regarding the zones where pounding occurs. Another conclusion that can be stated is that there are buildings for which pounding is much more damaging, such as the buildings situated in the extremities of a row of buildings or the ones exhibiting very different geometrical and dynamic characteristics to the neighbour ones.

KEYWORDS

Pounding; impact; structural gap.

INTRODUCTION

In recent earthquakes several cases of structural collapse have occurred in buildings which, in spite of having been designed according to current rules for seismic design, have experienced pounding with adjacent buildings.

Most of the world regulations for seismic design do not take into account this phenomenon and some of the ones who do it, do not provide specific rules that must be followed (Paz, 1994). Among the exceptions are the codes of Argentina, Australia, Bulgaria, Canada, France, Indonesia, Israel, Mexico, Peru, Taiwan, USA and Venezuela which specify a minimum gap size between adjacent buildings.

In some cases, the gap size depends only on the maximum displacements that each building may present. The rule to determine the gap size is nevertheless variable, being in some cases the simple sum of the displacements of each building (e.g. Canada and Israel) and in other cases a smaller value that may be either a percentage of the previous one (e.g. Peru) or a quadratic combination of the maximum displacements (e.g. France). In other cases the gap size is made dependent on the building height (e.g. Taiwan), in some cases a combination of the two rules is implemented and in others there is even a minimum gap size which varies between 2.5 cm (e.g. Argentina) and 15 cm (e.g. Taiwan).

In some codes, these values are made dependent on the type of soil and seismic action (e.g. Argentina) and in others on the structural typology, which is to say on the way the inelastic displacements are evaluated given the calculated elastic ones.

The search for simple rules to deal with pounding is thus centered on the aspect of the gap size, although the influence of some other parameters is also analysed.

STUDY METHODOLOGY

The model to analyse the buildings and its impact follows the generic rules of structural analysis programs, allows for the consideration of nonlinear behaviour of the structures and models impact by properly altering the dynamic equations of motion whenever the relative displacements between adjacent buildings exceed the available gap.

The behaviour of two adjacent structures is simultaneously analysed. Whenever the displacements between two given degrees of freedom (e.g. horizontal displacements of two nodes that may be subjected to impact) exceed the available initial gap, the quantities of movement corresponding to each one of the degrees of freedom subjected to impact are evaluated and new initial conditions for the movement are specified. These initial conditions are specified in terms of new velocities for each degree of freedom.

If the masses of the two degrees of freedom are respectively m_1 and m_2 and if their velocities just before impact are respectively v_1^i and v_2^i, than, the corresponding velocities after impact, v_1^f and v_2^f, may be evaluated as follows:

$$v_1^f = v_1^i - (1+e)\, m_2\, (v_1^i - v_2^i) / (m_1 + m_2)$$

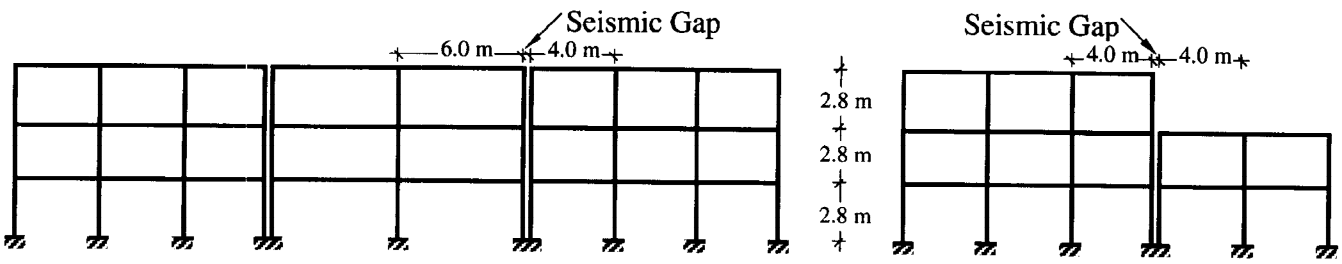
$$v_2^f = v_2^i + (1+e)\, m_1\, (v_1^i - v_2^i) / (m_1 + m_2)$$

where e is a coefficient of restitution (Hoppmann II, 1988), related to the type of impact. It varies between 0, when a totally plastic impact occurs and the kinetic energy is dissipated in terms of plastic deformation energy, and 1, corresponding to a totally elastic impact without variation of the total kinetic energy.

Previous studies (Mesquita, 1991) have suggested that global results are not very sensitive to this parameter and suggest that a value equal to 0.65 could be used for concrete structures. Thus, although the sensitivity of the results to this parameter was examined, this was the value used for most of the simulations.

Two different geometrical situations were examined. The first one (figure 1 a) consisting on a row of three buildings where the two exterior ones are identical. The exterior buildings have a fundamental natural frequency of 2.5 Hz and the interior one of 1.2 Hz. The interior building has a large mass lumped at the top story. In all structures the story floors were admitted rigid in the horizontal direction.

The second examined geometry (figure 1 b) corresponds to a situation of two adjacent buildings with different number of stories. The three stories building is identical to the external buildings of the previous geometry and the two stories building has a fundamental natural frequency of 5.5 Hz.



a) row of three buildings b) buildings with different height

Fig. 1 - Sets of buildings subjected to pounding

All buildings which are part of the above displayed sets were first supposed alone and subjected to a seismic loading corresponding to the specified in Eurocode 8 for a stiff soil. The structures were designed assuming a q factor equal to 3 and then subjected to artificial accelerograms in accordance to Eurocode 8, assuming nonlinear behaviour. The nonlinear structural analysis was performed using the stiffness degrading elements of DRAIN-2D, which follow a modified Takeda law. In all cases the following response parameters were evaluated: maximum displacements for all story levels, maximum interstory drifts for all story levels, maximum required ductility for all columns of a given story and maximum required curvature ductility for all beams of a given story.

To evaluate the effects of pounding, the buildings were then analysed considering pounding, and the same response parameters were evaluated and compared to the corresponding response parameters determined for the buildings alone. The increase or decrease in story displacements was analysed by means of a parameter which is the ratio between the observed maximum displacement with pounding (d_p^{max}) and the corresponding observed value for the building alone (d_a^{max}), $\lambda_d = d_p^{max} / d_a^{max}$.

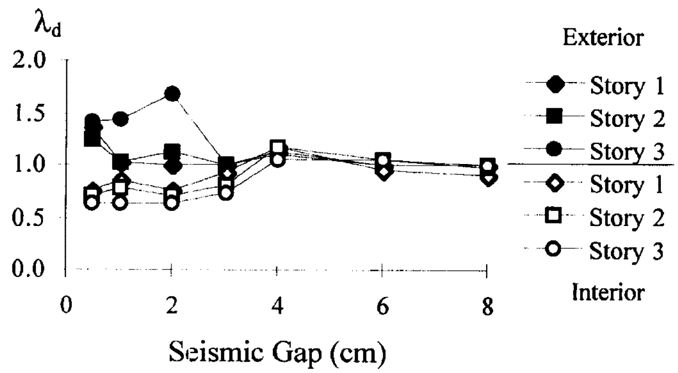
A similar parameter was used to assess the increase or decrease in the interstory drift ($\lambda_\Delta = \Delta_p^{max} / \Delta_a^{max}$). Values of λ_d or λ_Δ greater than 1 mean increase in displacements or interstory drift and smaller than 1 mean decrease of the same quantities.
In terms of maximum observed required ductility, other parameter was used to evaluate the influence of pounding, which represents the ratio between the required ductility increase, or decrease, and the required ductility observed in the building not subjected to pounding, $\lambda_D = (D_p^{max} - D_a^{max}) / D_a^{max}$. In this case, a positive value means a ductility increase and a negative value a ductility decrease. In all cases, if a linear behaviour was observed a ductility value equal to one was admitted.

The parameters whose influence on the different values of λ was examined are the gap size and the restitution coefficient e. Also studied was the influence of the seismic action, analysing the sensitivity of the response to five different accelerograms corresponding to the same earthquake spectrum, and the influence of the structural model, by comparing the results with the results obtained considering a linear behaviour for all structural elements.

RESULTS

The influence of the seismic gap on the controlling parameters can be seen in figures 2 to 5. In all these cases, a restitution coefficient e equal to 0.65 was adopted.

Figure 2 shows that the seismic gap has indeed a large influence on the amplification or reduction of displacements. For low seismic gaps there is a tendency to amplify the displacements of the exterior buildings and a tendency to diminish those of the interior building. λ_d values greater than 1 can be observed, for small seismic gaps, in all exterior building stories and specially in the top story, the opposite being true for all stories of the interior building. For seismic gaps larger than 3 cm, the tendency is for λ_d values close to 1, showing that pounding effects become less important. Similar conclusions can be taken from figure 3, which displays the λ_Δ values regarding the maximum interstory drift.



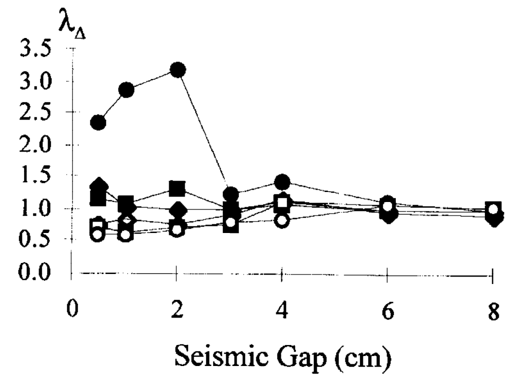
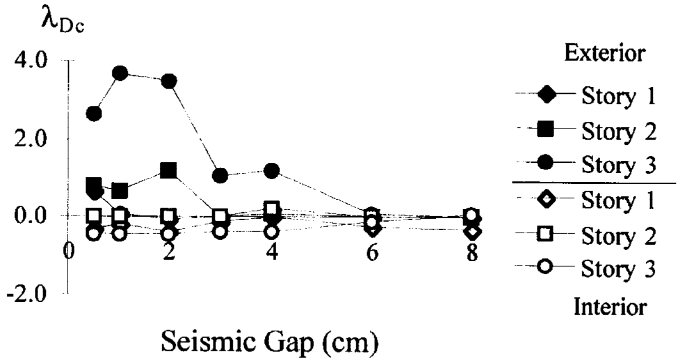


Fig. 2 - λ_d values as function of the seismic gap

Fig. 3 - λ_Δ values as function of the seismic gap

Figures 4 and 5 display the λ_D values respectively for columns (λ_{Dc}) nd beams (λ_{Db}). It can be observed a tendency for higher ductility demands in the exterior building, with a significant increase in the top story columns and a tendency for lower ductility demands in the interior building. It is worth noting that the maximum top displacements observed in the exterior and interior buildings when analysed alone are respectively 3.3 and 5.1 cm. Anyway, it can be observed that for gap sizes larger than approximately 6 cm, ductility requirements are comparable to the values observed in structures without pounding, suggesting that, in this case, this gap size could be acceptable, even being smaller than the sum of the maximum displacements.



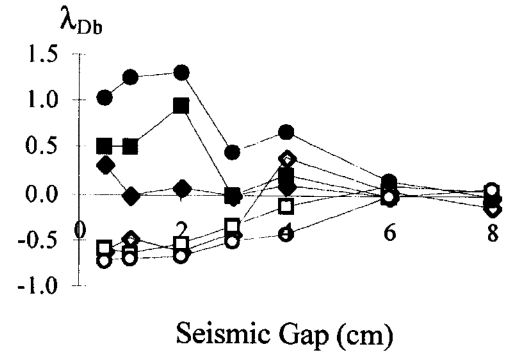


Fig. 4 - λ_{Dc} values as function of the seismic gap

Fig. 5 - λ_{Db} values as function of the seismic gap

Figures 2 to 5 show a clear distinction between the results obtained for the exterior and the interior buildings, the exterior ones being in some sense a kind of restraints of the interior ones, which is in agreement with previous studies performed using linear structural models (Mesquita, 1991). To assess the importance of considering nonlinear models, a comparison was made between the λ_Δ values obtained using linear and nonlinear models. This comparison is presented in figures 6 a and b respectively for the exterior and the interior buildings. It is clear, from figure 6, that assuming nonlinear models, the amplification due to pounding is much higher, specially for small gap sizes and in what regards the last story. Figure 7 shows that regardless of the gap size, the reduction observed in all story levels of the interior building is smaller if nonlinear behaviour is considered. This justifies the need to consider nonlinear behaviour whenever pounding phenomenon is present.

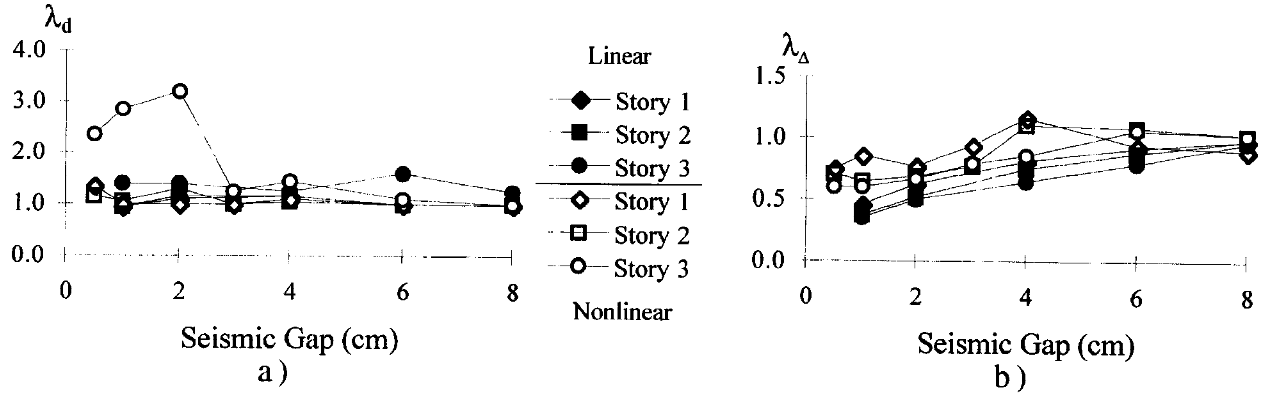


Fig. 6 - λ_Δ values obtained using linear and nonlinear models a) exterior building, b) interior building

Another parameter whose influence on the response needs to be examined is the restitution coefficient e. For that purpose, the already mentioned set of buildings was analysed considering different e values. Figures 7 to 10 show the influence of this parameter in all the λ values.

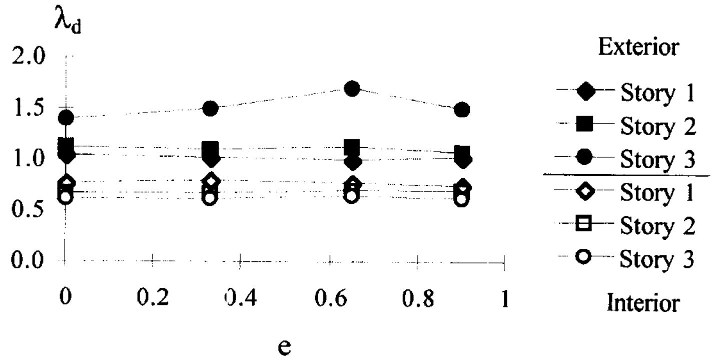


Fig. 7 - λ_d values as function of e

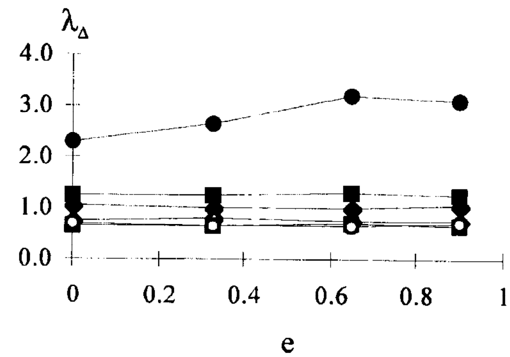


Fig. 8 - λ_Δ values as function of e

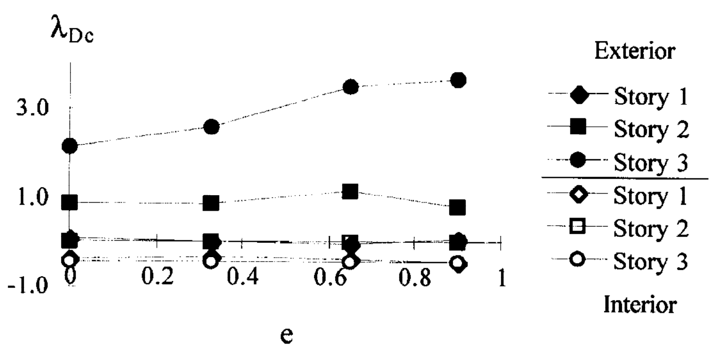


Fig. 9 - λ_{Dc} values as function of e

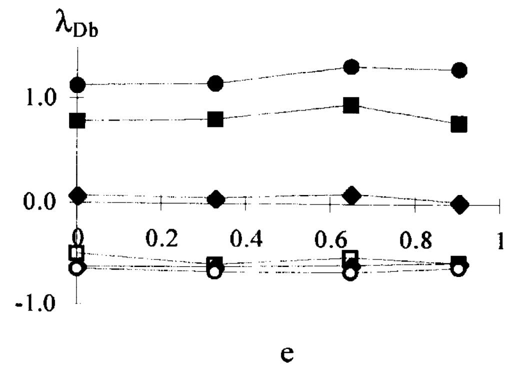


Fig. 10 - λ_{Db} values as function of e

It is clear, from these figures, that the variation in the restitution coefficient e has very limited influence on the response, except in what regards the response of the exterior building top story where, nevertheless, the influence is not very high and the higher values are obtained for e values ranging from 0.65 to 1.0. This observation justifies the use of e = 0.65 for the purpose of this or similar studies.

Another aspect that should be examined is the influence that the variability of the seismic motion, and more specifically in what regards the phases associated to each seismic wave, may have in the amplification or reduction of the structural response due to pounding effects. For that purpose, five different accelerograms were generated, all corresponding to the same power spectral density function but with different phases associated to each frequency wave. The results of the structural response parameters for each one of the generated time histories are presented in figures 11 to 14.

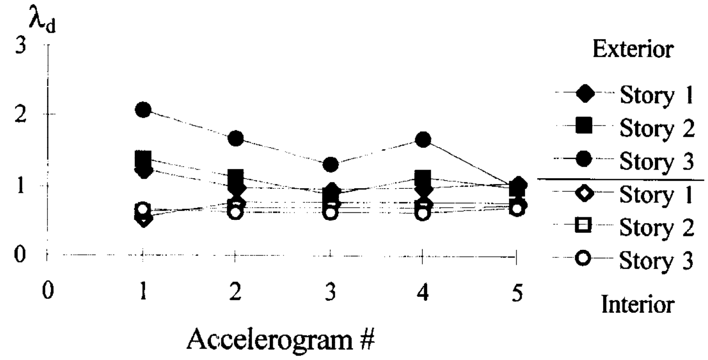


Fig. 11 - λ_d values as function of the seismic load

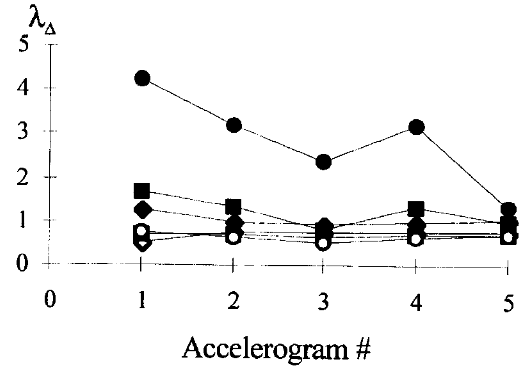


Fig. 12 - λ_Δ values as function of the seismic load

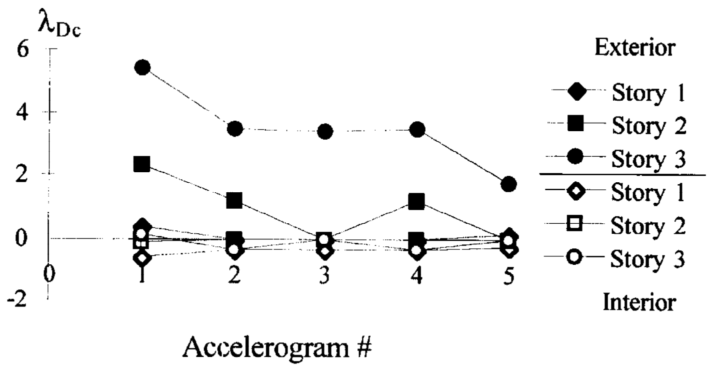


Fig. 13 - λ_{Dc} values as function of the seismic load

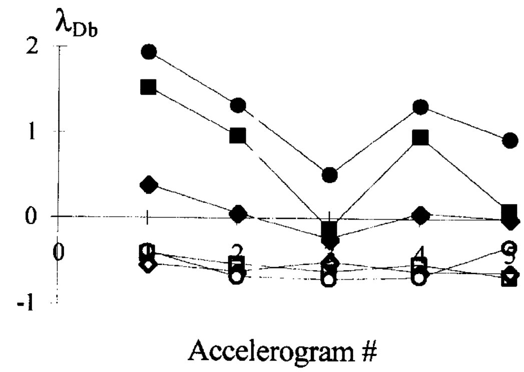


Fig. 14 - λ_{Db} values as function of the seismic load

The observation of these figures shows that the seismic load has a very large influence on all the λ values, justifying, similarly to other situations, the need to consider a large number of time histories whenever quantitative conclusions are to be expressed. Nevertheless it can be seen that in qualitative terms the variation in the seismic load does not change any one of the previous observations, namely the tendency to have larger λ values in the exterior building and in the top stories and reductions in λ for all interior building stories.

A different common situation of structural pounding occurs between buildings with different height. To analyse this situation, the set of two buildings displayed in figure 1 b was subjected to the same seismic loading and a procedure similar to the one used for the previous set of three buildings was followed.

Figures 15 to 18, display the influence of the gap size on all λ values. It can be seen that in this case the results do not vary for gaps larger than 3 cm. It is interesting to note that this gap size is approximately a quadratic combination of the maximum displacements experienced by the two structures at the second story level, when not subjected to pounding, which are respectively 2.4 and 1.3 cm for the left and right buildings. A similar finding had already been made regarding the previously analysed set of three buildings.It can also be seen that although the higher increase in story displacements (figure 15) occurs at the second story of the smaller structure, the higher increase in interstory drift (figure 16), and consequently in ductility (figures 17 and 18), occurs at the third story of the tallest structure and so above the point of impact. This can be explained by the fact that the bottom stories of the taller building are restrained by the smaller stiffer one and thus the top displacements of the taller building do not increase significantly in spite of the increase of the story drift above the level of impact.

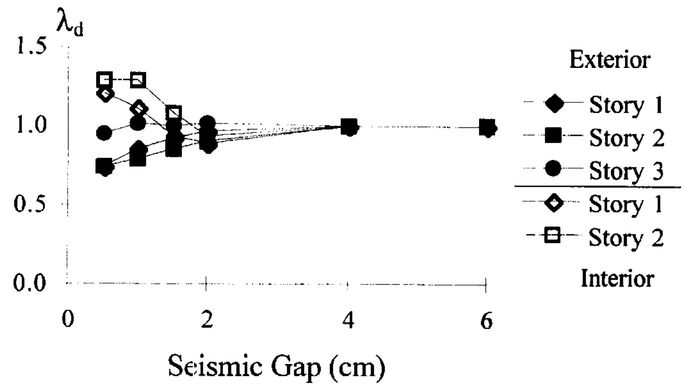


Fig. 15 - λ_d values as function of the gap size

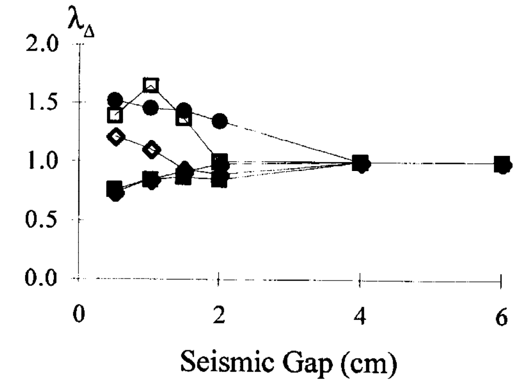


Fig. 16 - λ_Δ values as function of the gap size

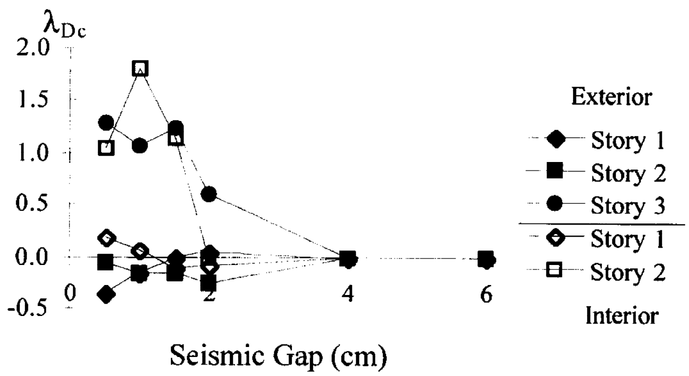


Fig. 17 - λ_{Dc} values as function of the gap size

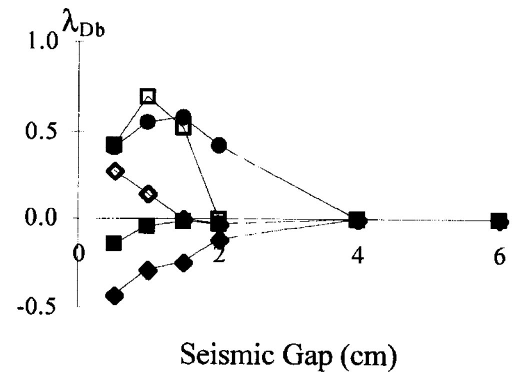


Fig. 18 - λ_{Db} values as function of the gap size

CONCLUSIONS

From the results obtained with the present study it is possible to state that the methodology used to model impact allows a correct simulation of pounding between adjacent buildings. Real time graphic visualisation of the structural response with pounding shows that the adopted procedure yields a very realistic representation of the pounding effects. If needed, when avoiding pounding is not possible, this methodology enables the designer to evaluate the pounding effects and design the structures in accordance.

The seismic gap size seems to be the most important parameter to prevent the negative consequences of pounding. From this point of view, and although the present results do not allow the establishment of a generic rule, it seems that it is not necessary to preview a seismic gap size equal to the sum of the maximum displacements of each building, as is specified in several codes. It seems enough to have a seismic gap size equal to the quadratic combination of the maximum displacements as foreseen in the French code. Anyway, special care should be taken to adequately evaluate those maximum displacements, being sure that they are the maximum nonlinear displacements corresponding to an earthquake used to verify the structural reliability for ultimate limit states.

Another important conclusion, which could be corroborated by results of previous studies (Mesquita, 1991), is that for buildings of the same height, the ones situated at the end of a row are always much more prone to withstand damage from pounding than the ones situated in the middle, which most of the times, even benefit from pounding. In this case the most affected zones are the top stories of the exterior buildings where large interstory drift and high ductility demand occur. This observation is quite important because could lead to rules of selective retrofitting of old buildings located in urban blocks, where the reinforcement would be preferentially done in the outer buildings.

For buildings with different height, the most affected zones are the zones where the smaller buildings hit the taller ones. Special care should thus be taken when designing taller buildings close to existing smaller and stiffer ones. In this case some preventive measures should preview a special reinforcement of the zones immediately below and above the zone of impact.

A pounding situation which has not been approached with this study is the impact of buildings having stories at different levels. This problem can not be treated in a comparable way to present used methodology because other phenomena such as the existence of localised high shear forces and even punching. This is, for sure, a much more dangerous situation, which should not be allowed and which justifies further studies.

REFERENCES

Hoppmann II, W. (1988). Effects of Impact on Structures, in *Shock and Vibration Handbook* (Harris C. editor), McGraw Hill, New York, USA.

Mesquita, C. (1991). *Impacto entre Estruturas Adjacentes devido à Acção Sísmica*. M. Sc. dissertation Instituto Superior Técnico, Lisbon, Portugal.

Paz, M. (editor) (1994). *International Handbook of Earthquake Engineering*. Chapman & Hall, New York, USA.