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Inter-story pounding between multistory reinforced concrete structures

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Abstract. The influence of the inter-story structural pounding on the seismic behaviour of adjacent multistory reinforced concrete structures with unequal total heights and different story heights is investigated. Although inter-story pounding is a common case in practice, it has not been studied before in the literature as far as the authors are aware. Fifty two pounding cases, each one for two different seismic excitations, are examined. From the results it can be deduced that: (i) The most important issue in the inter-story pounding is the local effect on the external column of the tall building that suffers the impact from the upper floor slab of the adjacent shorter structure. (ii) The ductility demands for this column are increased comparing with the ones without the pounding effect. In the cases that the two buildings are in contact these demands appear to be critical since they are higher than the available ductility values. In the cases that there is a small distance between the interacting buildings the ductility demands of this column are also higher than the ones of the same column without the pounding effect but they appear to be lower than the available ductility values. (iii) It has to be stressed that in all the examined cases the developed shear forces of this column exceeded the shear strength. Thus, it can be concluded that in inter-story pounding cases the column that suffers the impact is always in a critical condition due to shear action and, furthermore, in the cases that the two structures are in contact from the beginning this column appears to be critical due to high ductility demands as well. The consequences of the impact can be very severe for the integrity of the column and may be a primary cause for the initiation of the collapse of the structure. This means that special measures have to be taken in the design process first for the critically increased shear demands and secondly for the high ductility demands.

Key words: structural pounding; inter-story pounding; reinforced concrete structures; ductility requirements; non-linear dynamic analysis.

1. Introduction

Earthquake induced collisions between adjacent structures have been repeatedly reported in the literature as a usual case of damage. Based on reports of field observations after numerous destructive earthquakes all over the world, it can be concluded that pounding is frequently observed when strong earthquakes strike big cities and densely populated urban areas (Arnold and Reitherman 1982, Rosenblueth and Meli 1986, Bertero 1986). In these events it has been proved that the interaction between adjacent buildings is a usual cause of damage and moreover there are

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cases reported in literature that pounding has been identified as a primary cause for the initiation of collapse (Bertero 1986). In the event of the earthquake that struck Mexico City in 1985, damage due to pounding was identified in over 40% of 330 severely damaged or collapsed buildings and in 15% of all cases it led to collapse (Rosenblueth and Meli 1986, Bertero 1986). Although in this respect the earthquake of Mexico City is a unique case in terms of damage and collapse cases attributed to pounding, in all the earthquakes of the last decades structural pounding was always present. Furthermore, a resent survey of seismic separations between buildings for the Taipei City (Jeng and Tzeng 2000) reveals that for the 2359 surveying tall buildings 403 are predicted to suffer pounding damage. Among them 46 might collapse and 76 might suffer severe damage. It is emphasized that inter-story pounding which according to the authors it is known to cause instant collapse, would account for 39 of the cases of collapse or 85% of the total 46 cases.

During the last two decades many analytical investigations have been reported on the problem of the structural pounding. In the beginning these studies were based on the response of pairs or sets of colliding single degree of freedom systems in earthquake excitations. Results indicate that in the case of colliding alike systems in the row, exterior systems tend to suffer more due to the pounding effect than do the interior ones, the latter often experiencing reductions in their response. In the same direction the influence of a constant phase difference in the base motion of the base motion of colliding systems on the pounding effect is studied in an attempt to approximate the traveling wave effect (Athanasiadou *et al.* 1994).

Cases of pounding between multi-degree-of-freedom systems have been also examined. The buildings were idealized as lumped mass, shear beam type, multi-degree-of-freedom systems with bilinear force-deformation characteristics. The story levels of the colliding structures were always the same. Results of collisions on the response of a 5-story building in configurations of 2, 3 and 4 buildings in contact have been reported (Anagnostopoulos and Spiliopoulos 1992). Examination of the pounding effect in cases of two buildings with different number of stories is also included. In situations like these, according to the authors, pounding can be catastrophic.

Numerical formulations for the pounding of two structures focusing primarily on advanced solution techniques have also been reported during the past decade (Liolios 1990, Papadrakakis *et al.* 1991, Maison and Kasai 1990, 1992). Maison and Kasai (1990) proposed the formulation and the solution of the multiple degree of freedom equations of motion for a type of structural pounding between two buildings and presented the pounding between a tall 15-story structure and a shorter 8-story stiffer and more massive building. Formulation and results are based on elastic dynamic analysis. Chau and Wei (2001) have proposed a formulation to model pounding of two adjacent structures under harmonic earthquake excitation as non-linear Hertzian impact between two single-degree-of-freedom oscillators. Furthermore, Chau *et al.* (2003) have performed shaking table tests to investigate the pounding phenomenon between two single-degree-of-freedom steel towers of different frequencies and damping ratios subjected to different of stand-off distance and seismic excitation. In all the examined cases the story levels of the two colliding structures were always the same.

Karayannis and Fotopoulou (1998) examined various cases of structural pounding between multistory reinforced concrete structures designed according to the Eurocodes 2 and 8. The work is based on non-linear dynamic step-by-step analysis and its purpose was to present initial results for the influence of some critical pounding parameters on the ductility requirements of the columns and to examine the possibility of taking into account the pounding effect during the design process according to EC2 to EC8. In the examined cases the story levels of the two colliding structures were always the same.

The effect of soil flexibility on the inelastic seismic response of a particular case of adjacent 12- and 6-storey reinforced concrete moment-resisting frames are examined by Rahman *et al.* (2001).

Dynamic response of bridge systems with several simple spans can also experience pounding between adjacent decks. Kim *et al.* (2000) have concluded that pounding effects can cause remarkable changes in the seismic responses of adjacent vibration bridge units and they found that the effect of friction reduces the seismic response. Ruangrassamee and Kawashima (2003) propose for the control and mitigation of the pounding effect in bridge systems the use of variable dampers.

It is emphasized that all the previously mentioned papers examine pounding problems with buildings that have stories with equal inter-story heights and consequently the pounding takes place always between the floor masses of the colliding structures. Furthermore, these investigations are focused on displacements and ductilities whereas the shear demands and the shear capacity of the columns are totally neglected despite the fact that these parameters are also very important for the reinforced concrete structures that suffer structural pounding. Moreover, most of the existing analytical studies have yielded conclusions that are not directly applicable to the design of multistory buildings potentially under pounding.

Considering, also, that (i) pounding is a frequent cause of structural damage that under certain conditions it can lead to collapse initiation, (ii) the problem has not yet been studied effectively especially for the case of colliding structures with non-equal inter-story heights and (iii) according to the new design codes (Eurocodes 2 and 8, ACI 318) flexible frame structures prone to structural pounding can be designed; in this paper an attempt to study the influence of the structural pounding on the ductility and shear requirements for reinforced concrete structures with different story heights is presented. The examined structures are multistory reinforced concrete frames with unequal total heights and different story heights designed according to the codes EC2 and EC8. In these very common pounding cases the slabs of the diaphragms of the short stiffer structure hit the columns of the other structure at a point within the deformable height. This phenomenon is referred to as inter-story pounding and it can be considered, for obvious reasons, as the most critical case of earthquake induced interaction between adjacent multistory structures (Karayannis and Favvata 2005). Although inter-story pounding is a common case in practice (Arnold and Reitherman 1982, Jeng and Tzeng 2000), it has not been methodical investigated before in the literature as far as the authors are aware.

In this work fifty two pounding cases for two different seismic excitations are examined. The examined cases include pounding cases between multistory structures and a 3-story frame-wall structure or a rigid barrier. The heights of the story levels of the multistory structures are not the same with the ones of the stiffer 3-story structures. Furthermore, the influence of the size of the gap distance between the adjacent structures on the effects of the pounding is also investigated. Non-linear dynamic step-by-step analysis and special purpose elements are employed for the needs of this study.

2. Key assumptions

2.1 Model idealization of structural pounding

In this work the pounding between adjacent structures with different total heights and different story heights is studied. In these cases each structure responds dynamically and vibrates

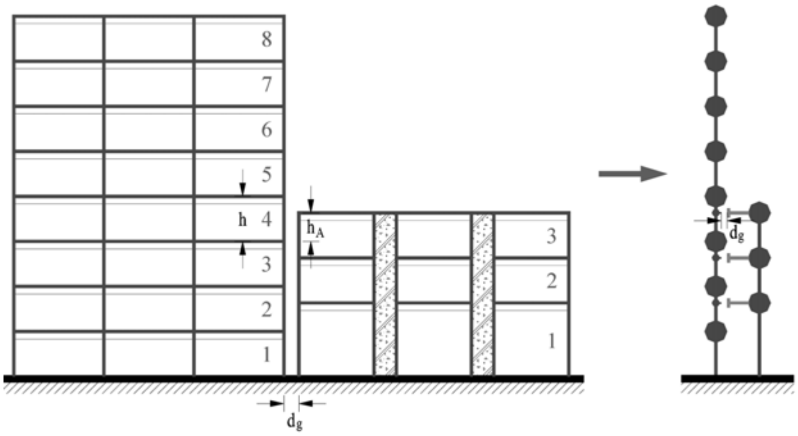


Fig. 1 Actual condition and model idealization of pounding problem. Structures with unequal total heights and the heights of the story levels of the two structures are not equal too. Pounding at the columns (interstory pounding)

independently. It is considered that one flexible multistory building is in contact or in close proximity to one less flexible shorter structure. If there is a gap distance between the structures collisions occur when the lateral displacements of the structures exceed the pre-defined gap distance (d_g). The influence of the gap size on the pounding effects is also investigated.

The heights of the story levels of the two structures are not equal (Fig. 1). In this very common case the slabs of the diaphragms of the shorter and stiffer structure hit the columns of the other structure at a point within the deformable height. This phenomenon is especially intense at the contact point of the upper story level of the short stiffer structure with the corresponding column of the tall building. Actual condition and the model idealization of this pounding case are shown in Fig. 1. Contact points are taking into account at the levels of the floor slabs of the short structure (Fig. 2d). Nevertheless, from the analyses of the examined pounding cases it has been found that the response of the interacting structures is influenced only by the position and the characteristics of the contact point at the short structure's top floor. The influence of the other contact points on the results proved to be negligible in the examined cases. The same conclusion also holds, more or less, for the examined cases with zero distance gap. This is mainly attributed to the significant height difference of the interacting structures in the studied cases. Thus, in the following analyses and results, only the influence of the pounding effect through the top floor contact point on the whole behaviour of the structures and on the response and ductility requirements of the columns is examined. Seismic analyses have been performed using time steps in the range of 1/5.000 sec to 1/10.000 sec in order to maintain equilibrium during the integration.

2.2 Contact element

Collisions are simulated using special purpose contact elements that become active when the corresponding nodes come into contact. This idealization is consistent with the building model used and appears adequate for studying the effects of pounding on the overall structural response for the pounding cases under examination. Local effects as inelastic flexural deformations, yield of the flexural reinforcement and ductility requirements of the columns in the pounding area are taken into

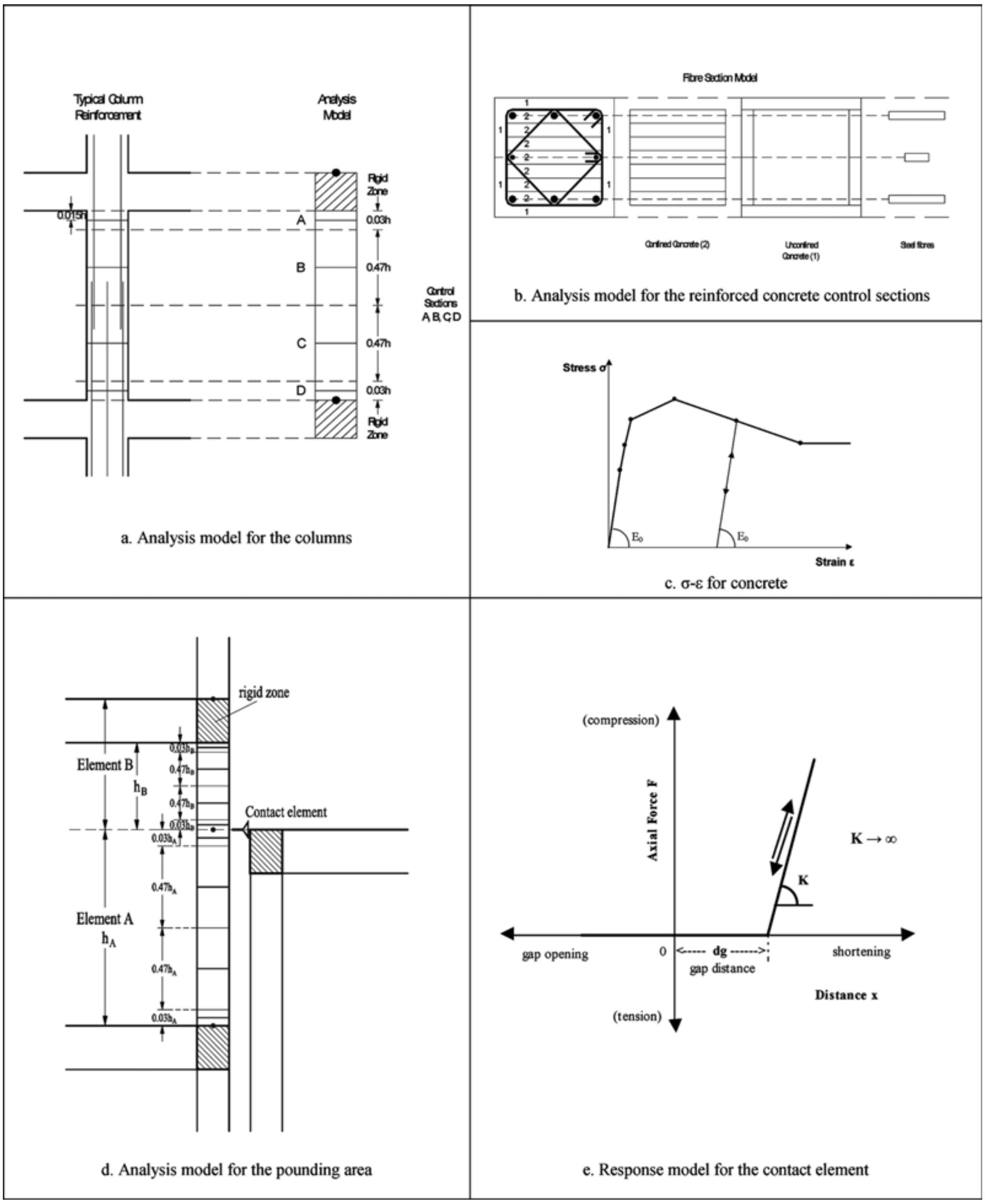


Fig. 2 Analysis model

account through the special purpose elements employed for the modeling of the columns.

The response of the contact elements is shown in Fig. 2(e). The negative direction of the X-axis represents the condition that the buildings move away from each other. In the positive direction of the X-axis there are two parts in order to simulate the actual behaviour of the structures in case there is a small gap distance (d_g) between them. It is possible that the structures move one towards the other but the displacements are small and the existing gap is not covered. In this case the contact element remains non-active and the buildings continue to vibrate independently. In the case that the structures move one towards the other and the displacements bridge the existing gap or the structures are in contact from the beginning then the contact element responds as a spring with large stiffness. The stiffness of the spring is typically large and highly uncertain due to the unknown geometry of the impact surfaces, the uncertain material properties under the impact loadings, the variable impact velocities etc. (Kim *et al.* 2000). Based on a sensitivity study it has been accepted that the system response is not quite sensitive to changes in the stiffness of the spring (Kim *et al.* 2000, Maison and Kasai 1992). In the case of inter-story pounding the damage at the contact area is expected to be concentrated from the beginning at the column that suffers the impact. Thus, considering that the damage of the building materials and the damage of the slabs of the shorter structure are not significant, a contact element without damping has been used. Moreover, analyses of pounding cases using contact elements that can account for damping have been performed as well. From comparisons between the results of these analyses with the results of the analyses using elastic contact elements, it was obtained that for the examined cases of inter-story pounding the observed differences were negligible.

2.3 Analysis model for structures and beam–column elements

The frame structural systems consist of beams and columns whereas the dual (frame-wall) systems have in addition two reinforced concrete walls. Each structure is modeled as a 2D assemblage of non-linear elements connected at nodes. The mass is lumped at the nodes and each node has three degrees of freedom. Each structure responds dynamically and vibrates independently. Collision occurs when the lateral displacements of the structures at the floor levels exceed the pre-defined gap distance (d_g) between the two structures.

The computer program used in this work is the program package DRAIN-2DX. The finite element mesh used here for the modeling of each structure utilizes an one-dimensional element for each structural member. Two types of one-dimensional beam-column elements were used: (a) One special purpose elements that is employed for the modeling of the columns of the multistory structure and (b) an element for the modeling of the beams of these structures and for all members of the second shorter structure. The latter element is a common lumped plasticity beam-column model that considers the inelastic behaviour concentrated in zero-length "plastic hinges" at the element's ends.

The special purpose element employed for the columns is one of "distributed plasticity" type accounting for the spread of inelastic behaviour both over the cross-sections and along the deformable region of the member length. This element performs numerical integration of the virtual work along the length of the member using data deduced from cross-section analysis at pre-selected locations. Thus, the deformable part of the element is divided into a number of segments and the behaviour of each segment is monitored at the centre cross-section (control section) of it. The cross-section analysis that is performed at the control sections is based on the fibre model. This fibre model accounts rationally for axial – moment (P-M) interaction. Thus, the hysteretic behaviour of the columns is mainly based on the hysteretic rule of the materials used in the analysis. The fibre section model and the stress-strain ($\sigma - \varepsilon$) relationship for the concrete used in this work are shown in Figs. 2(b) and (c), respectively.

In order to accurately model the actual behaviour of the columns the deformable height of each column is divided into four segments. The lengths of the segments have been determined such that the length of the inner two equal segments are equal to 0.47 h whereas the length of the segments at the ends of the element are equal to 0.03 h, where h is the deformable height of the column (Fig. 2a). This partition of the column's deformable height can reasonably take into account without excessive increase of the computational effort the following important structural parameters for the behaviour of a reinforced concrete column: (a) The actual distribution of the quantity of the longitudinal reinforcement along the column length. (b) The variation of the confinement degree of concrete over the cross sections and along the length of the column since higher degree of confinement is usually applied near the element's ends. (c) This partition of the column's length also allows for the setting of control sections near the element's ends very close to the face of the joints. These parts are considered to be critical zones because they are areas of potential formation of plastic hinges.

The confinement degree of the concrete can be represented by the coefficient K. The coefficient K is defined as:

$$K = \frac{f_{cc}^*}{f_{cc}}$$

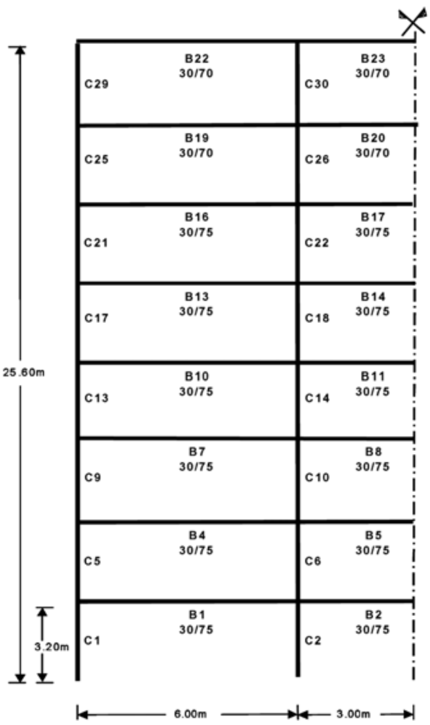
f_{cc} : compressive strength of unconfined concrete
f_{cc}^* : compressive strength of confined concrete
The confinement coefficient K actually represents the influence of the effectiveness of the confinement and the mechanical volumetric ratio of the confining reinforcement on the compressive strength of the concrete. In order to take into account the distribution of the confinement degree of concrete along the length of the columns the appropriate value for the coefficient K is defined for each control section. For the multistory structures designed according to the Eurocode 2 (2002) & Eurocode 8 (2003), meeting the Ductility Capacity Medium (DCM) criteria, the confinement degree in the middle part of the internal columns was rather low. The confinement coefficient was ranged from K = 1.023 to 1.041, while in the ending parts of the same (internal) columns was ranged from K = 1.213 to 1.305. However, regarding the external columns according to the Eurocodes 2 & 8 the confinement rules for the critical regions of columns are applied for the entire length of the members. Thus, for the external columns of the DCM multistory structures, the degree of confinement was the same for the entire length and the confinement coefficient was ranged from K = 1.213 to 1.309.

Furthermore, in this work special attention has been given for the study of the local response of the column that suffers the direct hit of the upper slab of the shorter and stiffer structure. In this direction, two special purpose elements of "distributed plasticity" type are employed for this column. Each element is divided in four unequal segments the way it is shown in Fig. 2(d). Thus, there are eight control cross-sections along the height of the critical column. This partition of the column's deformable height can reasonably take into account the actual distribution of reinforcement and confinement degree of concrete and further it allows for the setting of the control cross-sections near the element's critical points.

3. Design of structures

3.1 Multistory structures

Four multistory frame structures have been designed for the purposes of this work; two 8-story



COLUMNS of 8story frame structures designed by EC2& EC8					
INT.	**DCM FRAME**	**DCH FRAME**	**EXT.**	**DCM FRAME**	**DCH FRAME**
C2	**50/50 cm**	**55/55**	**C1**	**50/50**	**50/50**
	up 8Ø20 mm dn 12Ø20	10Ø20		up 8Ø20 dn 8Ø20 +2Ø14	8Ø20
C6	**50/50**	**50/50**	**C5**	**50/50**	**50/50**
	10Ø20	8Ø20		8Ø20	8Ø20
C10	**45/45**	**50/50**	**C9**	**45/45**	**45/45**
	8Ø20 + 4Ø18	8Ø20		4Ø20 + 4Ø16	4Ø20 + 4Ø16
C14	**45/45**	**45/45**	**C13**	**35/35**	**40/40**
	8Ø20 + 2Ø16	4Ø20 + 4Ø16		8Ø20 + 2Ø14	4Ø20 + 2Ø16
C18	**40/40**	**40/40**	**C17**	**30/30**	**35/35**
	12Ø20	up 4Ø20 + 6Ø16 dn 6Ø20 + 6Ø16		4Ø20 + 4Ø18	4Ø18 + 2Ø16
C22	**40/40**	**40/40**	**C21**	**30/30**	**30/30**
	up 10Ø20 dn 4Ø20 + 6Ø14	up 4Ø20 + 4Ø16 dn 4Ø20 + 2Ø18		4Ø20 + 2Ø16	6Ø14
C26	**35/35**	**35/35**	**C25**	**30/30**	**30/30**
	up 6Ø20 dn 4Ø20 + 4Ø16	4Ø18 + 2Ø12		4Ø20 + 2Ø14	up 6Ø14 dn 4Ø16 + 2Ø14
C30	**35/35**	**35/35**	**C29**	**30/30**	**30/30**
	8Ø14	4Ø18 + 2Ø12		6Ø14	6Ø14

INT.: Internal columns
EXT.: External columns

Fig. 3 Structural system and columns reinforcement of the 8-story frames designed to EC2 & 8

frame structures, one 6-story structure and one 12-story structure. Both 8-story frame structures were designed according to Eurocodes 2 and 8, the first one meeting the Ductility Capacity Medium (DCM) criteria and the latter one the Ductility Capacity High (DCH) criteria of these codes. The other two structures were designed according to Eurocodes 2 and 8 too, both meeting the Ductility Capacity Medium (DCM) criteria. Behaviour factors for DCM and DCH frames were $q = 3.75$ and 5.00, respectively. The mass of the structures is taken equal to $M = (G + 0.3Q)/g$ (where G gravity loads and Q live loads) and the design base shear force was equal to $V = (0.3\ g/q)M$ (where q the behaviour factor of the structure). Reduced values of member moments of inertia (I_{ef}) were considered in the design to account for the cracking; for beams $I_{ef} = 0.5\ I_g$ (where I_g the moment of inertia of the gross section) and for the columns $I_{ef} = 0.9\ I_g$. Critical for the dimensioning of the columns proved to be in most of the cases the code provision regarding the axial load ratio limitation $v_d \leq 0.65$ and $v_d \leq 0.55$ for DCM and DCH frames, respectively, and in a few cases (columns of the upper stories) the code requirements for minimum dimensions. Structural geometry and reinforcement of the columns of the 8-story frames are shown in Fig. 3. Furthermore, modal shapes and modal periods of the 8-story DCM frame structure are shown in Fig. 4(a).

3.2 Three-story structure

One 3-story is also designed according to the codes EC2 and EC8, meeting the DCM design criteria. The structure is a dual (frame-wall) structural system. The behaviour factor is taken equal to 3.0. The mass is taken equal to $M = (G + 0.3Q)/g$ and the design base shear force was equal to

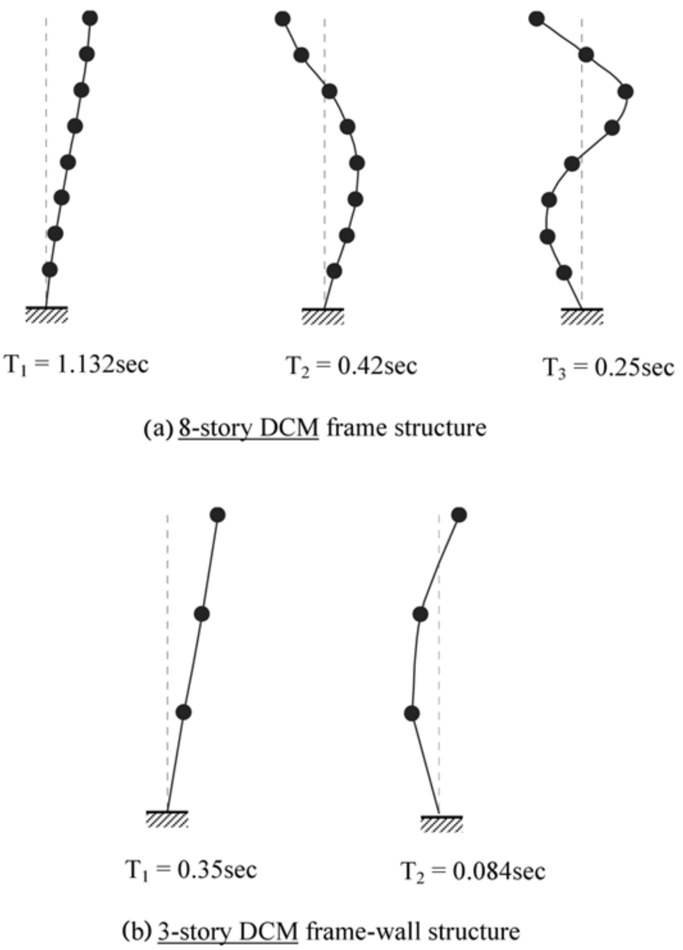


Fig. 4 Modal shapes and modal periods for the 8-story frame and the 3-story frame-wall structures

$V = (0.3\ g/q)M$. Reduced values of member moments of inertia (I_{ef}) were considered in the design; $I_{ef} = 0.5\ I_g$, $I_{ef} = 0.9\ I_g$ and $I_{ef} = 2/3\ I_g$ for beams, columns and walls, respectively. Critical for the dimensioning of the columns proved to be in most of the cases the code minimum requirements. The height of the first storey is 4.80 m and the height of the other stories is 3.20 m. Modal shapes and modal periods of the 3-story DCM frame-wall structure are shown in Fig. 4(b).

4. Description of the pounding cases

Fifty two (52) pounding cases, each one for two earthquake excitations are examined in this work (Table 2). These cases are sorted into three series; each series includes the pounding cases of the multistory frame with the 3-story structure and the 3-story rigid barrier. Thus, series A includes the pounding cases of the 8-story frames (DCM frame and DCH frame), series B includes the pounding cases of the 6-story frames and series C includes the pounding cases of the 12-story frame. Series A includes the pounding cases of two 8-story frames with the 3-story structure and the 3-story rigid barrier and it has been proved that these cases yielded the most interesting results.

Each one of the pounding cases of all the series is examined for two different gap distances (d_g) between the two structures and is analyzed for two seismic excitations. In these pounding cases the total height of the 3-story structure or 3-story barrier is greater than the total height of the 3rd floor and less than or equal to the total height of the 4th floor of the multistory frame and, thus, in the first case the contact point of the two structures lies between the levels of the 3rd and the 4th floor of the multistory frame and in the second case at the 4th floor. In the cases that the contact point lies between the levels of the 3rd and 4th floor it is considered that the pounding takes place at points of the deformable height of the columns of the more flexible multistory frame structure (inter-story pounding). Thus, each pounding case is examined for three positions of the contact point:

(a) The highest contact point is located at the 1/3 of the interstory height (h) of the column of 4th floor of the multistory frame (h_A = 1/3 h, see Figs. 1 and 2d).
(b) The highest contact point of the two structures lies between the levels of the 3rd and the 4th floor of the multistory frame at the 2/3 of the height of the column of the 4th floor (h_A = 2/3 h, see Figs. 1 and 2d).
(c) The total height of the 3-story structure is equal to the height of the 4th floor of the multistory frame. In this case pounding takes place between the mass of the 4th floor of the multistory frame and the mass of the 3rd floor of the shorter structure. This is not a case of inter-story pounding but is included in this series for comparison reasons.

Furthermore, each pounding case is subjected to two different natural seismic excitations;

Earthquake	Magnitude	Station	Component	PGA (g)	Duration
a. Imperial Valley (19/5/1940)	M_S 7.2	117 El Centro Array #9	I-ELC180	0.318	15 secs
b. Korinth, Greece (24/2/1981)	M_L 6.6	Korinth	NS	0.306	12 secs

The maximum acceleration values (PGA) of these excitations are very close to the design acceleration of the examined structures (PGA = 0.3 g).

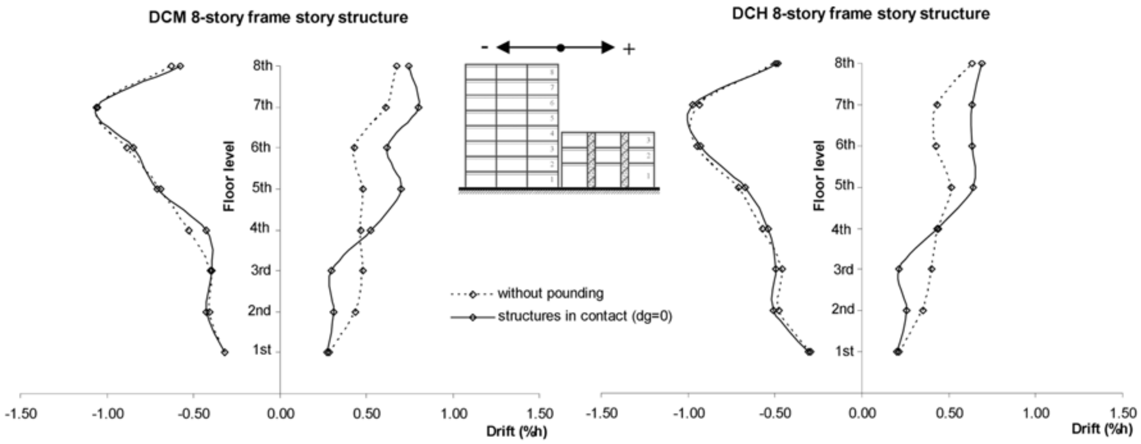
5. Results and comparisons

The results and the conclusions deduced from the analyses of each series include the observed overall response of the multistory frame and the ductility requirements of its columns whereas special attention has been given to the response of the 4th story column of the multistory frame where the pounding takes place. Furthermore, results for the shear demands and shear capacity of the column that suffers the hit are also presented and commented uppon.

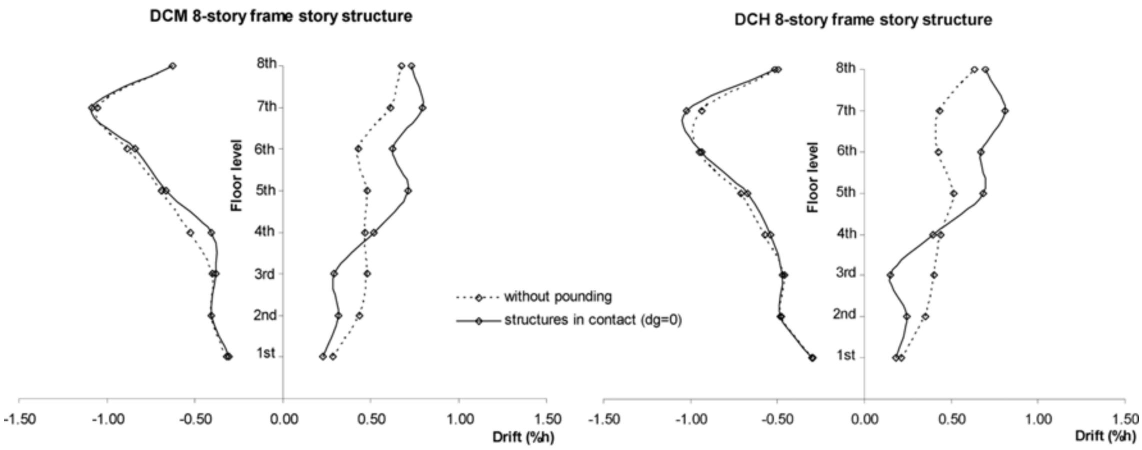
5.1 Series A

5.1.1 Overall response - Drifts - Ductility requirements

The maximum interstory drifts of the pounding cases of the 8-story frames (DCM and DCH



(a) Interstory pounding at the point h_A=1/3h of the 4th floor between 8-story frames and 3-story structure. Interstory drift.



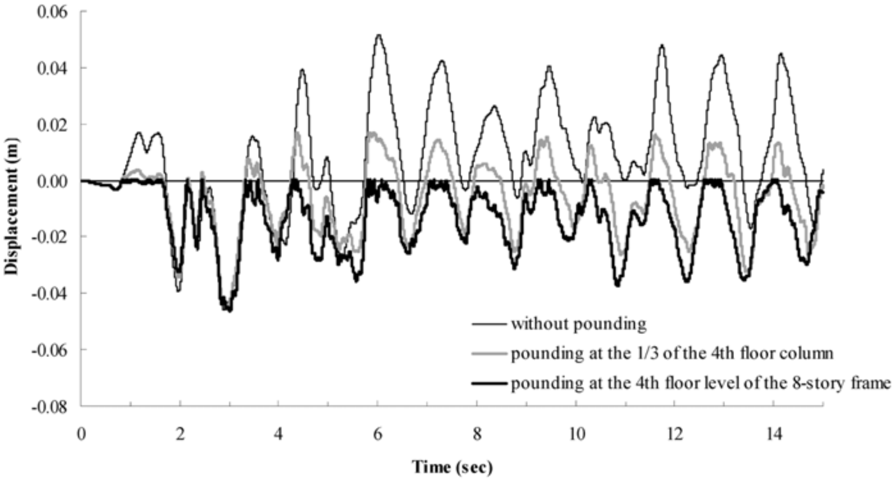
(b) Interstory pounding at the point h_A= 2/3h of the 4th floor between 8-story frames and 3-story structure. Interstory drift.

Fig. 5 Interstory pounding between 8-story and 3-story structures. Maximum Interstory drifts of the 8-story structures

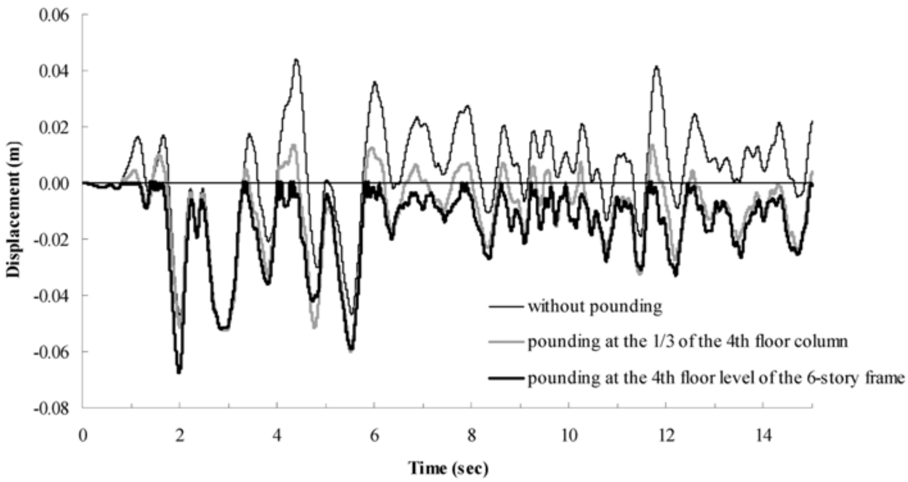
frames) with the 3-story structure are presented in Fig. 5 and are compared in the same figure with the ones of the 8-story frames vibrating without pounding effects. Figs. 5(a) comprise the pounding cases that the pounding takes place at the point h_A = 1/3 h of the 4th floor of the 8-story frame, whereas Figs. 5(b) comprise the cases that the pounding takes place at the point h_A = 2/3 h of the 4th floor of the 8-story frame.

The displacement time histories of the 4th floor of the 8-story DCM frame for the pounding between the 8-story frame and the 3-story rigid structure for the cases:

a) pounding takes place at the 4th floor
b) pounding takes place at the 1/3 of the height of the 4th floor column of the 8-story frame are presented in Fig. 6(a) and are compared with the time history of the 4th floor of the 8-story frame when it vibrates alone (without pounding effect).



(a) Time history of the 4th level of the 8-story DCM frame structure



(b) Time history of the 4th level of the 6-story DCM frame structure

Fig. 6 Time history results of pounding cases between multistory frames and the 3-story rigid structure. Structures in contact from the beginning ($d_g = 0$) (for the seismic excitation Imperial Valley, 1940)

The response of the 3rd floor of the 3-story frame-wall structure is also examined. In particular the displacement time histories of the 3rd floor of the 3-story frame-wall structure for the pounding case between the 8-story DCM frame and the 3-story structure when pounding takes place at the 1/3 of the height of the 4th floor column of the 8-story frame is shown in Fig. 7 and is compared with the displacement time history of the same floor when the 3-story structure vibrates alone (without pounding effect). Furthermore, for comparison reasons in the same figure displacement time history of the same floor (3rd) of the 3-story structure for the case that pounding takes place at the 4th floor of the 8-story frame is also shown.

The curvature ductility requirements for the columns of the 8-story DCM frame for the cases that the pounding takes place (a) at the point $h_A = 1/3$ h and (b) at the point $h_A = 2/3$ h of the 4th floor of

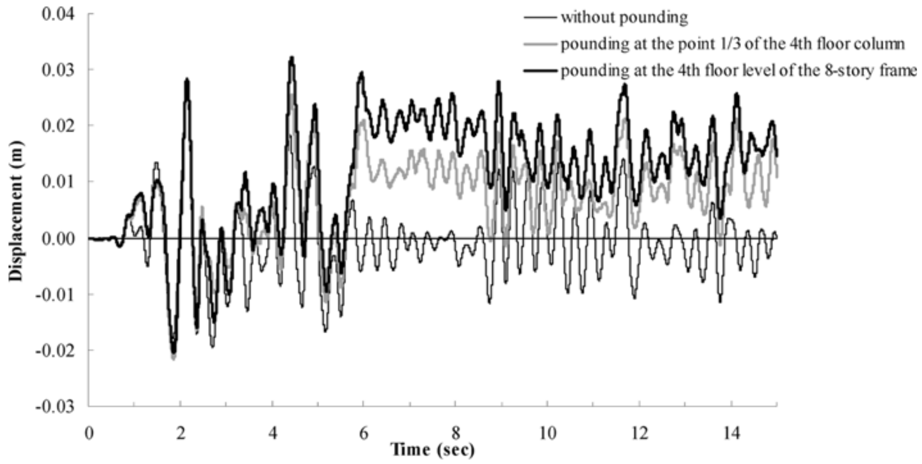


Fig. 7 Time history results of the 3rd level of the 3-story DCM frame-wall structure. Structures in contact from the beginning ($d_g = 0$) (for the seismic excitation Imperial Valley, 1940)

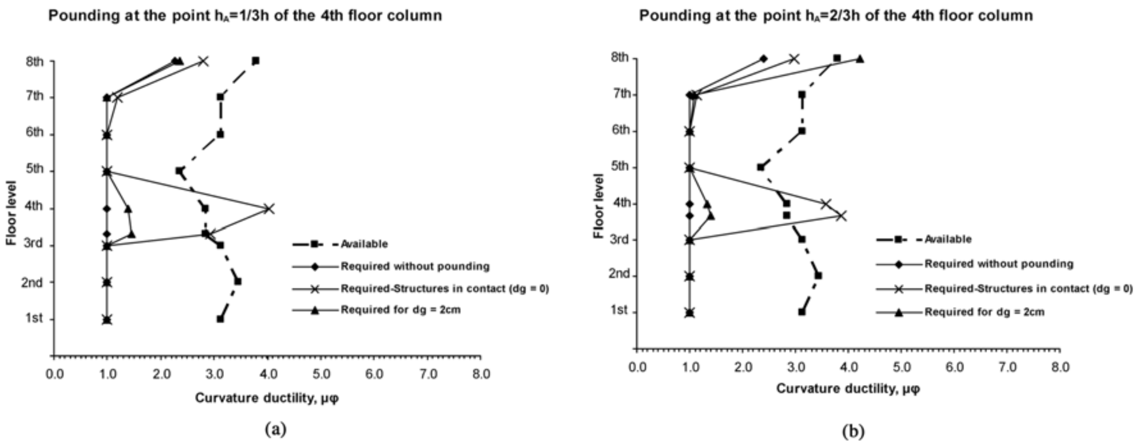


Fig. 8 Interstory pounding between the 8-story DCM frame and the 3-story structure. Ductility requirements of the external columns at the pounding side of the 8-story frame

the 8-story frame are presented in Fig. 8 for the pounding cases (i) the 8-story frame and 3-story structure are in contact from the beginning ($d_g = 0$), (ii) there is a gap distance equal to 2 cm ($d_g = 2$ cm) between the colliding structures from the beginning and (iii) the 8-story frame vibrates without pounding effects. In theses figures the ductility requirements are compared with the available values of curvature ductility of the columns as designed according to the codes EC2 and EC8. Similar results for the columns of the same frame for the pounding cases of the DCM frame with the 3-story stationary barrier are presented in Fig. 9.

Furthermore, results for the columns of the 8-story DCH frame for the pounding cases of the 8-story DCH frame with the 3-story structure are presented in Fig. 10. In this figure results are presented only for the case $d_g = 0$ since for the case $d_g = 2$ cm the results have shown that the ductility requirements of the columns were not affected substantially by the pounding.

Based on these results it is observed that the maximum interstory drifts (Fig. 5) and the ductility

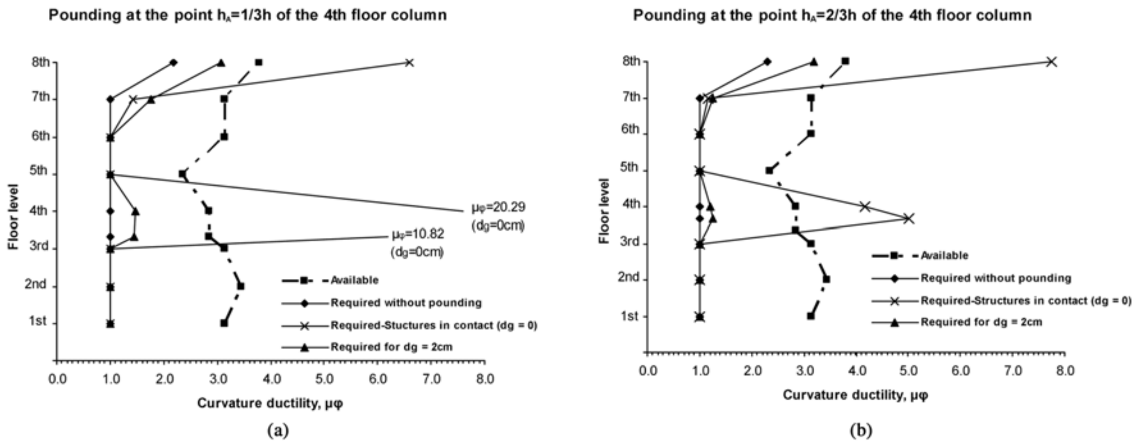


Fig. 9 Interstory pounding between the 8-story DCM frame and the 3-story stationary barrier. Ductility requirements of the external columns at the pounding side of the 8-story frame

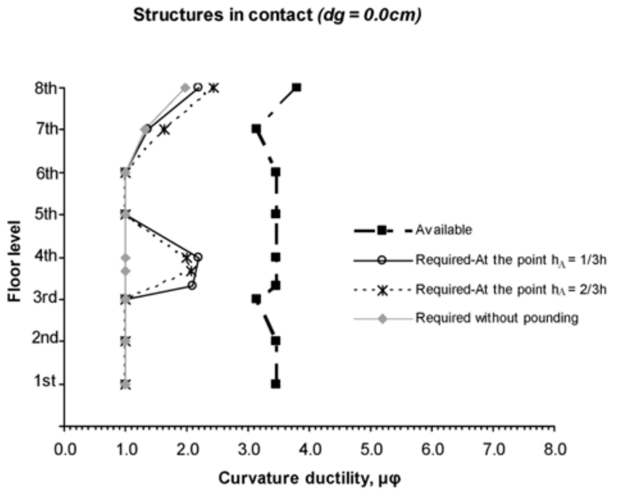


Fig. 10 Interstory pounding between the 8-story DCH frame and the 3-story structure. Ductility requirements of the external columns at the pounding side of the 8-story frame

requirements of the columns of the 8-story frames (Figs. 8, 9 and 10) are increased for the floors above the floor of the contact (4th floor) in comparisons with the ones of the same frames without the pounding effect. This is probably attributed to a whiplash type of behaviour of the taller structure. The whiplash type of behaviour becomes especially intense in the pounding cases between the 8-story frame and the 3-story stationary barrier. In this extreme case the curvature ductility requirements of the upper floor columns of the 8-story frame have exceeded the available curvature ductilities.

From the results it can be seen that in the cases that there is a gap between the adjacent structures ($d_g = 2.0$ cm) ductility requirements are reduced compared with the ones in the cases that structures are in contact from the beginning ($d_g = 0$).

5.1.2 Pounding area

The most important issue in the examined pounding cases is obviously the local effect on the external column of the tall building that suffers the impact from the upper floor slab of the adjacent shorter and stiffer structure. This impact takes place at a point of the deformable height of the column. The consequences of the impact can be very severe for the integrity of the column and may be a primary cause for the initiation of the collapse of the structure. This is probably the most critical case of interaction between adjacent buildings and although it is a common case in practice it has not been studied before in the literature as far as the authors are aware.

In this work special attention has been given for the study of the local response of the structural member of the multistory frame that suffers the hit from the upper floor slab of the adjacent shorter and stiffer structure. Results concerning the flexural and the shear demands of this column are presented and compared with the corresponding available values for the examined pounding cases.

For the pounding case of the 8-story DCM frame with the 3-story structure the ductility requirements of the column that suffers the hit are presented in Figs. 8(a) and 8(b), for the cases of pounding at the points $h_A = 1/3$ and 2/3 of the column height (h), respectively. From these figures it can be observed that the ductility demands for the column that suffers the pounding impact (4th story column) are increased when compared with the ones without the pounding effect and especially for the cases that the two buildings are in contact from the beginning ($d_g = 0$) these demands appear to be higher than the available ductility values. In the cases that there is a small gap distance ($d_g = 2$ cm) between the interacting buildings the ductility demands of the column are also higher than the ones of the same column without the pounding effect (Fig. 8) but they appear to be lower than the available ductility values.

Ductility requirements of the column that suffers the hit for the pounding case of the 8-story DCH frame with the 3-story structure for the cases of pounding at the points $h_A = 1/3$ and 2/3 of the column height (h) are presented in Fig. 10. In this figure results are presented for the structures in contact from the beginning ($d_g = 0$).

Furthermore, ductility demands for the column that suffers the hit for all the examined pounding cases are presented in Table 1. From this table it can be observed that the ductility demands are substantially increased for the cases of pounding with the 3-story rigid barrier. Moreover, it is noted that for the pounding cases with the rigid structure and for $d_g = 0$ the ductility demands always exceed the available values (Table 1).

The developing shear forces of the critical part of the column that suffers the impact for the pounding case of the DCM and DCH 8-story frames with the 3-story structure are presented in the Figs. 11(a) and 11(b) for the case of pounding at the point $h_A = 1/3$ h of the column that suffers the hit, where h the column height. In these figures results are presented for the case that the two structures are in contact from the beginning ($d_g = 0$) and for the case that there is a gap between the two structures equal to 2 cm ($d_g = 2$ cm). In these figures each point represents the developing shear force, V, and the axial force, N, at a step of the seismic analysis, whereas the lateral solid lines show the available capacity of the reinforced concrete element for the combination of shear versus axial force (EC2 & 8). This way a direct comparison of the developing shear force at the steps of the analysis with the available shear strength can be obtained. It can be observed that in all the examined cases the developing shear forces exceed the shear strength of the column many times during the excitation.

Analyses results for the pounding cases between the 8-story DCM frame and a 3-story stationary barrier that represents a very stiff not vibrating structure, are presented in Fig. 9. The ductility

Table 1 Interaction between multistory frames and the 3-story structure or the rigid barrier. Curvature ductility demands of the multistory frame external column that suffers the inter-story pounding

DUCTILITY DEMANDS Multistory Frames - External Column That Suffers the Pounding						
Series	Without pounding	With pounding				
A - <u>8-story</u> DCM-frame	Elastic	Pounding with:	3-story structure (η_T = 3.23)*		3-story rigid barrier ($\eta_T \rightarrow \infty$)*	
		Pounding at:	d_g = 0.0 cm	d_g = 2.0 cm	d_g = 0.0 cm	d_g = 2.0 cm
		h_A = 1/3 h	4.03 (1.41)+	1.45(0.51)+	20.29 (7.11)+	1.48 (0.52)+
		h_A = 2/3 h	3.86 (1.35)	1.39(0.49)	5.02 (1.76)	1.26 (0.44)
		4th floor	Elastic		Elastic	
DCH-frame	Elastic	Pounding with:	3-story structure (η_T = 3.27)		3-story rigid barrier ($\eta_T \rightarrow \infty$)	
		Pounding at:	d_g = 0.0 cm	d_g = 2.0 cm	d_g = 0.0 cm	d_g = 2.0 cm
		h_A = 1/3 h	2.20 (0.64)	1.02 (0.30)	9.00 (2.61)	2.35 (0.68)
		h_A = 2/3 h	2.08 (0.60)	1.02 (0.30)	9.22 (2.67)	2.55 (0.74)
		4th floor	1.06 (0.31)	elastic	1.84 (0.53)	2.59 (0.75)
B - <u>6-story</u> DCM-frame	Elastic	Pounding with:	3-story structure (η_T = 2.35)		3-story rigid barrier ($\eta_T \rightarrow \infty$)	
		Pounding at:	d_g = 0.0 cm	d_g = 2.0 cm	d_g = 0.0 cm	d_g = 2.0 cm
		h_A = 1/3 h	6.06 (1.76)	elastic	7.40 (2.14)	1.10 (0.32)
		h_A = 2/3 h	1.07 (0.31)	elastic	17.79 (5.15)	1.26 (0.36)
		4th floor	elastic	elastic	1.84 (0.53)	1.21 (0.35)
C - <u>12-story</u> DCM-frame	Elastic	Pounding with:	3-story structure (η_T = 3.70)		3-story rigid barrier ($\eta_T \rightarrow \infty$)	
		Pounding at:	d_g = 0.0 cm	d_g = 2.0 cm	d_g = 0.0 cm	d_g = 2.0 cm
		h_A = 1/3 h	Elastic			
		h_A = 2/3 h				
		4th floor				

*$\eta_T = T_1/T_2$ where T_1 : Period of the multistory structure, T_2 : Period of the 3-story structure
+Ratio of the ductility demand to the available one

requirements of the column that suffers the hit for two seismic excitations for the cases of pounding of the 8-story frame with the stationary barrier at the points h_A = 1/3 h and h_A = 2/3 h, where h of the height of the 4-story column, are presented in Figs. 9(a) and 9(b), respectively. From these figures it can be observed that the ductility demands for the column that suffers the pounding (4th story column) are increased when compared with the ones without the pounding effect. In the cases that the two buildings were initially in contact (d_g = 0) these demands appear to be very high and in both cases higher than the available ductility values. It is also stressed that in these cases the developing

Table 2 Pounding cases – Examined parameters

Multistory Frames - Examined Parameters*			
Number of stories n_s	Pounding with	Gap distances	Contact point
a. 6-story DCM **b. 8**-story DCM **c. 8**-story DCH **d. 12**-story DCM	frame – wall structure	$d_g = 0.0$ cm	4th floor $h_A = 1/3$ h $h_A = 2/3$ h
		$d_g = 2.0$ cm	4th floor $h_A = 1/3$ h $h_A = 2/3$ h
	rigid barrier (very stiff structure)	$d_g = 0.0$ cm	4th floor $h_A = 1/3$ h $h_A = 2/3$ h
		$d_g = 2.0$ cm	4th floor $h_A = 1/3$ h $h_A = 2/3$ h
Also: Response without pounding for 6-story DCM, 8-story DCM, 8-story DCH, 12-story DCM			

*All cases are examined for two seismic excitations (Imperial Valley and Korinth)

shear force of the critical part of the column that suffers the impact exceeded the shear strength of the column many times during the seismic excitation.

Thus, in the cases where inter-story pounding may take place special measures for the columns that will suffer the hit have to be taken in the design process aiming to meet the critically increasing shear and ductility demands due to the potential pounding effect. Also, special measures as the strengthening of the columns and the joints that will suffer the hit (Tsonos 2002) have to be taken for existing structures under potential pounding due to recently built new adjacent structures.

5.2 Series B

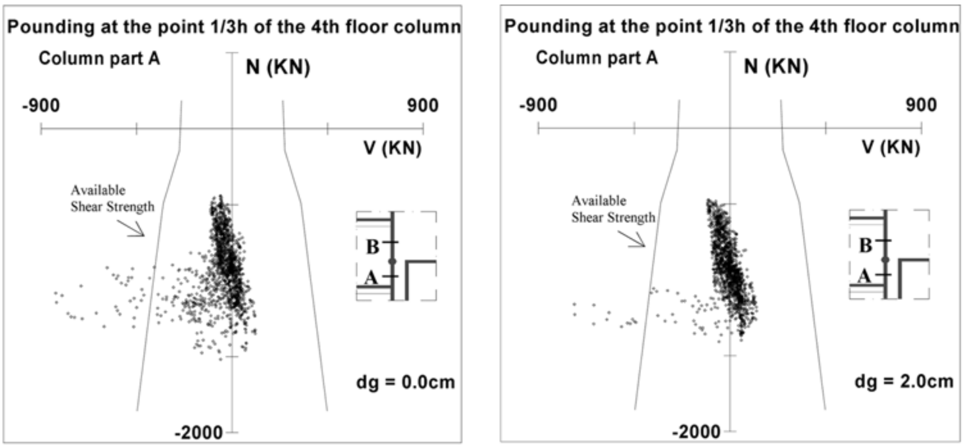
The pounding cases between the 6-story frame and the 3-story structure and the cases between the 6-story frame and the 3-story rigid barrier are included in series B. The displacement time histories of the 4th floor of the 6-story DCM frame for the pounding between the 6-story frame and the 3-story rigid structure for the cases:

a) pounding takes place at the 4th floor

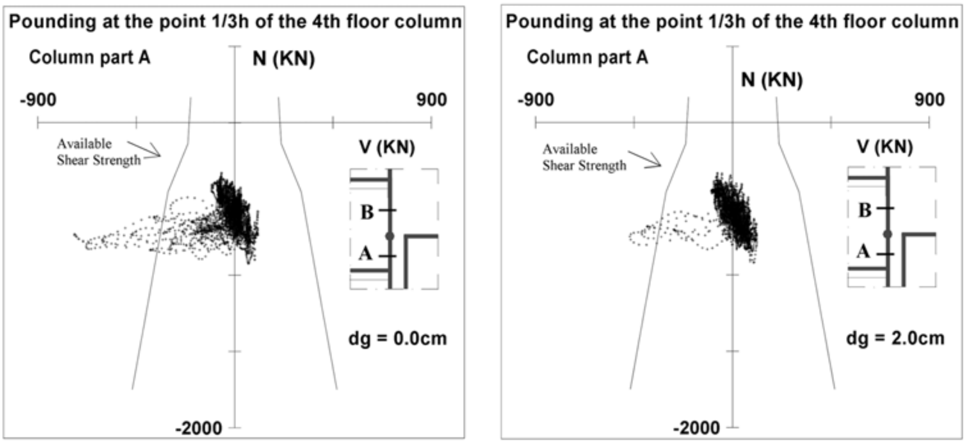
b) pounding takes place at the 1/3 of the height of the 4th floor column of the 6-story frame are presented in Fig. 6(b) and are compared with the time history of the 4th floor of the 6-story frame when it vibrates alone (without pounding effect).

From the pounding cases examined in this series it can be deduced that the ductility requirements of the columns of the 6-story frame are not substantially increased due to the pounding effect except for the column of the 4th floor that suffers the hit of the top floor slab of the 3-story structure.

Ductility requirements of the external columns (pounding side) of the 6-story frame for the pounding case between the 6-story frame and the 3-story structure and for the case of pounding at the point $h_A = 1/3$ h of the column height (h), are presented in Fig. 12(a). In this figure results are



(a) Interstory pounding between 8-story DCM frame and 3-story structure.



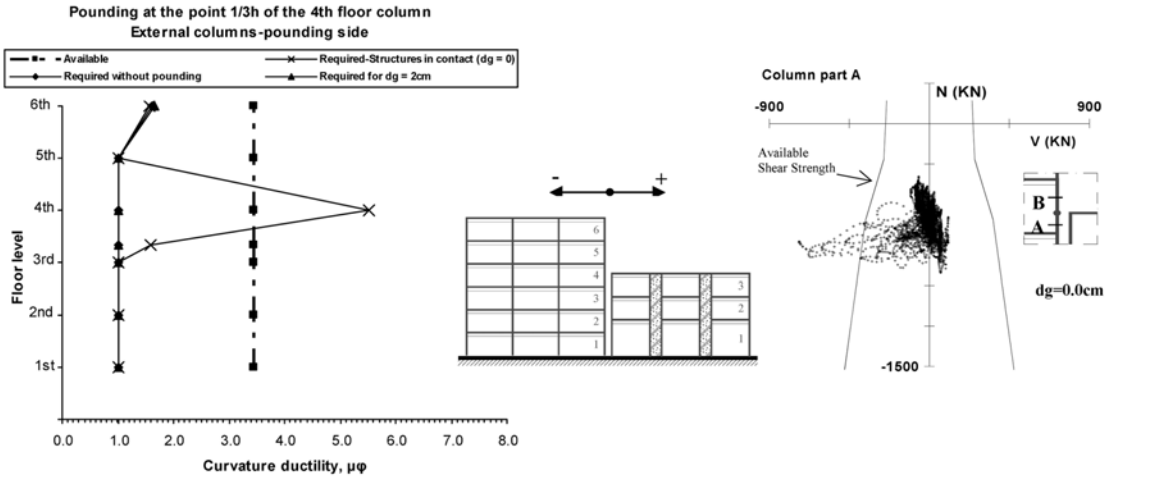
(b) Interstory pounding between 8-story DCH frame and 3-story structure.

- Each point represents the pair of the developing Shear force and Axial force at a step of the seismic analysis

N : Axial force , V : Shear force

Fig. 11 Interstory pounding between the 8-story and the 3-story structure at the point $h_A = 1/3$ h of the 4th floor column. Shear forces developing in critical column (lower part A) of the 4th story of the 8-story frame and available strength

presented for the case that the structures are in contact from beginning ($d_g = 0$) and for the case that there is an initial gap distance between the two structures equal to $d_g = 2$ cm. These results are compared with the ductility requirements of the 6-story structure for the case that the structure vibrates without pounding effect as well as with the available ductility as deduced from cross-section analyses of the reinforced concrete columns. Based on these results it can be observed that the ductility demands are increased compared with the ones of the structure vibrating without pounding effect only for the column that suffers the pounding effect for the case that the two structures are in contact from the beginning ($d_g = 0$). It is noted that in this case the ductility



(a) Ductility requirements of the external columns at the pounding side of the 6-story frame.

(b) Shear forces developing in critical column of the 4th story of the 6-story frame and available strength.

Fig. 12 Interstory pounding between the 6-story and the 3-story structure at the point $h_A = 1/3$ h of the 4th floor column

demands for the column that suffers the hit exceed the available ductility.

Similar results have been obtained for the pounding cases between the 6-story frame and the 3-story rigid structure. Although in these cases the pounding phenomenon was more intense only the ductility demands for the column that suffers the hit exceeded the available ductility in the case that the structures were in contact from the beginning (Table 1).

Results concerning the developing shear forces have shown that the shear strength demands are also critical for the cases of interaction between the 6-story frame and the 3-story structures (Fig. 12b) as well as for the cases of pounding between the 6-story frame and the 3-story rigid barrier.

5.3 Series C

The pounding cases between the 12-story frame and the 3-story structure and the cases between the 12-story frame and the 3-story rigid barrier are included in series C. From the pounding cases examined in this series it can be deduced that the ductility requirements of the columns of the 12-story frame are not substantially increased due to the pounding even for the column of the 4th floor that suffers the hit of the top floor slab of the 3-story structure.

Ductility requirements of the external columns (pounding side) of the 12-story frame for the pounding case between the 12-story frame and the 3-story structure have been compared with the ductility requirements of the 12-story structure for the case that the structure vibrates without pounding effect as well as with the available ductility as deduced from cross-section analyses of the reinforced concrete columns. Based on these results it can be obtained that the ductility demands were not increased compared with the ones of the structure vibrating without pounding effect. It is noted that in all the examined cases the ductility demands for all columns including the column that suffers the hit have not exceeded the available ductility. Similar results have been obtained for the pounding cases between the 12-story frame and the 3-story rigid structure. Although in these cases

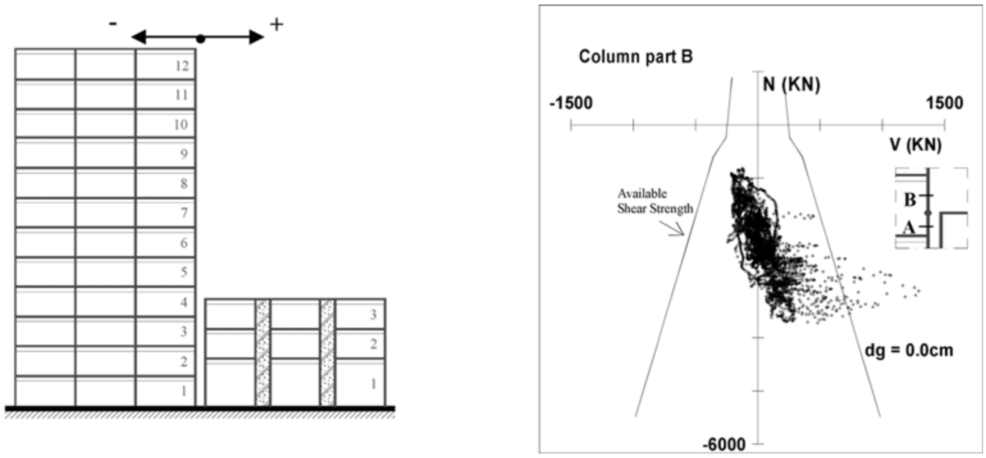


Fig. 13 Interstory pounding between the 12-story and the 3-story structure at the point h_A = 2/3 h of the 4th floor column. Shear forces developing in critical column (upper part B) of the 4th story of the 12-story frame and available strength

the interaction phenomenon was more intense the ductility demands for all the columns of the 12-story frame including the column that suffers the hit have not exceeded the available ductility (Table 1).

Nevertheless, results concerning the demands for shear force strength, unlike the ductility demands, have shown that the shear strength demands are critical for the cases of pounding between the 12-story frame and the 3-story structure as well as for the cases of pounding between the 12-story frame and the 3-story rigid barrier (Fig. 13).

5.4 Effect of the number of stories on the response

From the results (Table 1) it can be seen that small increase of the number of stories of the tall building (from six to eight stories) has greatly influenced the observed demands for ductility and shear strength of the columns that suffer the hit (see also Table 1 and Figs. 11 and 12). These demands in the pounding cases of the 8-story frame with the 3-story structure (case with $d_g = 0$) have been decreased in comparison with the demands observed in the pounding cases of the 6-story frame with the 3-story structure (case with $d_g = 0$). The examined pounding cases of the 8-story frame with the 3-story very rigid structure yielded in some cases increased demands and in other cases decreased demands for ductility and shear strength in comparison with the demands observed in the corresponding pounding cases of the 6-story frame with the 3-story very rigid structure. Thus, no safe conclusions can be extracted about the influence of small changes of the number of stories on the demands for ductility and shear strength.

Nevertheless, high increase of the number of stories of the tall building (from 6 to 12 stories) in all the examined pounding cases has decreased substantially the demands for ductility and shear strength of the columns that suffer the hit, up to the point that the pounding effect did not really affect these demands (see Table 1 and Figs. 11, 12 and 13).

6. Conclusions

An investigation of the influence of the structural pounding on the seismic behaviour of adjacent multistory reinforced concrete structures with unequal total heights and different story heights is presented. In these very common pounding cases the slabs of the diaphragms of the short stiffer structure hit the columns of the other structure at a point within the deformable height. This phenomenon is referred to as inter-story pounding and it is considered as the most critical case of earthquake induced interaction between adjacent multistory structures. Although inter-story pounding is a common case in practice, it has not been studied before in the literature as far as the authors are aware.

The following conclusions can be deduced based on results of the examined cases: (i) Most important issue in the inter-story pounding cases is the local response of the external column of the tall structure that suffers the impact of the upper floor slab of the adjacent shorter and stiffer structure. The consequences of the impact can be very severe for the integrity of the column and may be a primary cause for the initiation of the collapse of the structure. (ii) It has been observed that the ductility demands for the column that suffers the pounding hit are substantially increased comparing with the ones when the structure vibrates without the pounding effect. In the cases that the two buildings are in contact these demands appear to be critical since they are higher than the available ductility values. In the cases that there is a small gap distance ($d_g = 2$ cm) between the interacting buildings the ductility demands of this column are also higher than the ones of the same column when the structure vibrates without the pounding effect but they appear to be lower than the available ductility values. (iii) It has to be stressed that in all the examined cases the observed shear forces of the critical part of the column that suffers the impact, exceed the shear strength of the column.

In general, the existence of a gap between the adjacent structures decreases drastically the high ductility demands of the columns that suffer the hit but it is not equally effective in decreasing the developing shear forces due to the pounding.

From the parametric study reported in this work it has been deduced that no safe conclusions can be extracted about the influence of small changes of the number of stories of the tall building on the demands for ductility and shear strength of the columns that suffer the hit. High increase, though, of the number of stories of the tall building (from 6 to 12 stories) in all the examined pounding cases has decreased substantially the demands for ductility and shear strength of the columns that suffer the hit, up to the point that the pounding effect did not really affect these demands.

Thus, it can be concluded that in the inter-story pounding cases the column that suffers the impact is always in a critical condition due to shear action and, furthermore, in the cases that the two structures are in contact this column appears to be critical due to high ductility demands as well.

This means that in cases where inter-story pounding may take place special measures for the columns that will suffer the hit have to be taken in the design process aiming to meet primary the critically increasing shear demands and secondary the high ductility demands due to potential pounding effect. Finally, it can be summarized that in situations where potentially inter-story pounding is neglected it may lead to non-conservative or even hazardous building design or evaluation.

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