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Influence of exterior joint effect on the inter-story pounding interaction of structures

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Abstract. The seismic induced interaction between multistory structures with unequal story heights (inter-story pounding) is studied taking into account the local response of the exterior beam-column joints. Although several parameters that influence the structural pounding have been studied sofar, the role of the joints local inelastic behaviour has not been yet investigated in the literature as key parameter for the pounding problem. Moreover, the influence of the infill panels as an additional parameter for the local damage effect of the joints on the inter-story pounding phenomenon is examined. Thirty six interaction cases between a multistory frame structure and an adjacent shorter and stiffer structure are studied for two different seismic excitations. The results are focused: (a) on the local response of the critical external column of the multistory structure that suffers the hit from the slab of the adjacent shorter structure, and (b) on the local response of the exterior beam-column joints of the multistory structure. Results of this investigation demonstrate that the possible local inelastic response of the exterior joints may be in some cases beneficial for the seismic behaviour of the critical column that suffers the impact. However, in all the examined cases the developing demands for deformation of the exterior joints are substantially increased and severe damages can be observed due to the pounding effect. The presence of the masonry infill panels has also been proved as an important parameter for the response of the exterior beam-column joints and thus for the safety of the building. Nevertheless, in all the examined inter-story pounding cases the presence of the infills was not enough for the total amelioration of the excessive demands for shear and ductility of the column that suffers the impact.

Keywords: structural interaction; inter-story pounding; exterior joints local damage effect; infill panels; non-linear seismic analysis.

1. Introduction

During the last two decades many analytical investigations have been reported addressing 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. In the same direction the influence of a constant phase difference in the base motion of the base motion of

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colliding systems on the pounding effect is studied in an attempt to approximate the traveling wave effect (Athanasiadou *et al.* 1994).

Few cases of pounding between multi-degree-of-freedom systems have been 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 1992). Examination of the pounding effect in cases of two buildings with different number of stories is also included (Karayannis and Favvata 2005a, b). In situations like these, 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, Liolios *et al.* 1991, Papadrakakis *et al.* 1991, Maison and Kasai 1990, Maison and Kasai 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-storey structure and a shorter 8-storey 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 almost always the same.

Karayannis and Fotopoulou (1998) examined various cases of structural pounding between multistory reinforced concrete structures designed according to the Eurocodes 2 (EC2) and 8 (EC8). 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.

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.

Karayannis and Favvata (2005a) for the first time, among others (Liolios *et al.* 1991), examined the influence of the structural pounding problem on the ductility requirements and the overall seismic response of reinforced concrete structures with unequal heights (inter-story pounding) designed according to the EC2 and EC8. Results of seventy two pounding cases between structures with equal inter-story heights and each one for two different seismic excitations were presented and used in order to quantify the pounding effect. Moreover, in this work initial results for the pounding case between adjacent structures where the slabs of the one structure hit the columns of the other one were included.

Recently, an extensive investigation on the pounding problem between multistory reinforced concrete frames with unequal total heights and different story heights designed according to the codes EC2 and EC8 was presented by Karayannis and Favvata (2005b). Fifty two pounding cases each one for two different seismic excitations were examined. In these cases the slabs of the diaphragms of a short stiffer structure hit the columns of another structure at a point within the deformable height and this phenomenon was referred to as inter-story pounding. The effect of the number of stories on the response of the multistory frame structures that suffers the pounding effect was also investigated. The most important problem in the case of inter-story pounding of reinforced concrete structures yielded to be the developing critical shear state at the columns that suffer the hit, since in these cases the demands of flexural ductility can more safely satisfied.

In the previous mentioned papers several parameters that influence the structural pounding have been investigated. However, considering that the brittle failure of reinforced concrete members is inherent critical situation for the seismic performance of the structures, the role of the beam-column joints possible inelastic local behaviour has not been included sofar as key parameter for the pounding problem in the literature as the authors are aware of. It is noted that the inelastic response and the stiffness deterioration of the reinforced concrete (RC) beam-column joints have been already recognized as important parameters that have to be considered carefully in the seismic analysis of structures especially in cases of old buildings. Obviously, modern codes do require shear reinforcement in the joint core areas, nevertheless, older existing building stock constitutes the main part of the center of the big cities in Europe. These kind of buildings have very small amount of shear reinforcement or no stirrups at all in the joint areas even in high seismic zones. On the other hand the structural pounding usually takes place in big city-centres where these old buildings exist.Thus, the study of a structural model without other major design deficiencies but only the lack of stirrups in the joints is a rational choice for these kind of well constructed old multistory RC buildings that may also suffer the interaction pounding effect.

It is generally accepted that the reinforced concrete (RC) beam-column connection can show a brittle behaviour when severe damage is concentrated within the joint core area. Over the last four decades, a large number of experimental and analytical studies have been carried out in order to investigate the seismic performance of the RC beam-column connections and to further identify their failure mechanisms. Several approaches with various degrees of accuracy have been presented for modelling the inelastic response of the joints, ranging from empirical methods to finite element models (Biddah and Ghobarah 1999, Youssef and Ghobarah 2001, Lowes and Altoontash 2003, Shin and Lafave 2004, Fleury *et al.* 1999). Moreover, the last few years the attention of some researchers has been focused on including the deformability of the RC beam-column connections due to slippage on the evaluation of the RC frame structures seismic performance (Coronelli and Mulas 2001, Cosenza *et al.* 2006). Although limited, studies that include the influence of the joints local effect on the non-linear seismic analysis of complete RC multistory frame structures have also been reported in the literature (Ghobarah and Biddah 1999, Calvi *et al.* 2002, Favvata *et al.* 2008).

Based on the above mentioned remarks in this work the effect that the beam-column joints strength and stiffness degradation has on the seismic response of a multistory RC frame structure that suffers the inter-story structural pounding is investigated. The local damage of the RC exterior joints is under primary consideration, since their effect has been repeatedly identified as leading cause of failure and eventual collapse of reinforced concrete buildings (Paulay and Priestley 1992). Recently, a special purpose model for the simulation of the exterior beam-column joints local response has been developed and implemented in a general program for non-linear static and dynamic analysis of structures as a spring joint element (Favvata 2006, Favvata *et al.* 2008).

Damages in old designed RC buildings during the recent earthquakes also indicated that the interaction between un-reinforced masonry infills and bare frame can lead to unexpected effects on the seismic response of the RC building such as shear failure in columns, damage to joint region and soft storey mechanism. Thus, considering that brittle failures can be occurred by the concentration of high stresses transferred from the infills to both joint and columns the interaction between masonry infill and bare frame should also be considered in the seismic analysis of structures especially in case of old buildings. Based on these thoughts the study of the influence of the infill panels on the earthquake interaction between adjacent structures is also attempted in this work.

Thirty six pounding cases for two different seismic excitations are studied. The examined cases include pounding cases between an 8-storey structure and a 3-storey frame-wall structure or a rigid barrier. The heights of the story levels of the multistory structure are not the same with the ones of the stiffer 3-storey structures. Five different positions of the contact point within the deformable height of the columns that suffer the hit are considered. In order to incorporate the influence of the joints local damage effect on the examined inter-story pounding cases a special purpose element (Favvata 2006, Favvata *et al.* 2008) is employed in the finite analysis mesh of the structural systems. Two types of beam-column joints behaviour are considered for the need of this study: (a) all exterior joints are considered with reduced capacity and simulated using the recently proposed model, and (b) all the joints are assumed as rigid ones. Furthermore, the influence of the size of the initial gap distance between the adjacent structures on the effects of the pounding relevant to the exterior joints possible local damage is also investigated. Finally, the influence of the masonry infill panels on the seismic response of the critical column of the multistory structure and on the seismic response of the exterior joints that suffers the pounding effect is also included taken into account three types of infilled multistory structures: (a) bare frame structure (without infills), (b) fully infilled structure and (c) infills are considered in the 2nd-8th stories and thus soft 1st story (pilotis building).

The results are focused on the local response of the critical column of multistory structure that suffers the hit from the slab of the adjacent shorter structure and on the exterior joints local response.

2. Key assumptions

2.1 Model idealization of structural pounding – contact element

The pounding problem that is studied in this work involves the interaction case between adjacent structures with different total heights and different story heights. It is considered that one flexible multistory building is in contact or in close proximity to one less flexible shorter structure. If there is an initial gap distance between the structures collisions occur when the lateral displacements of the structures exceed the pre-defined gap distance (d_g).

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

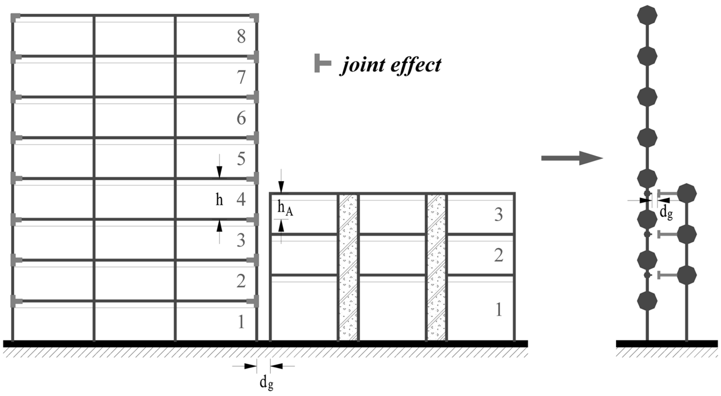


Fig. 1 Actual condition and model idealization of inter-story pounding problem that includes exterior joints local effect

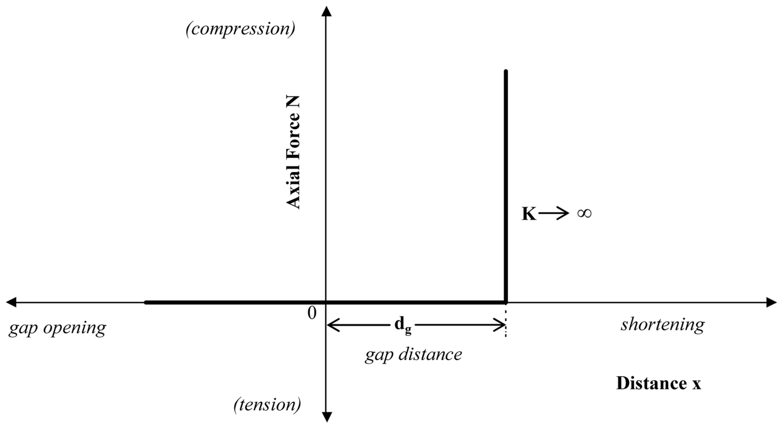
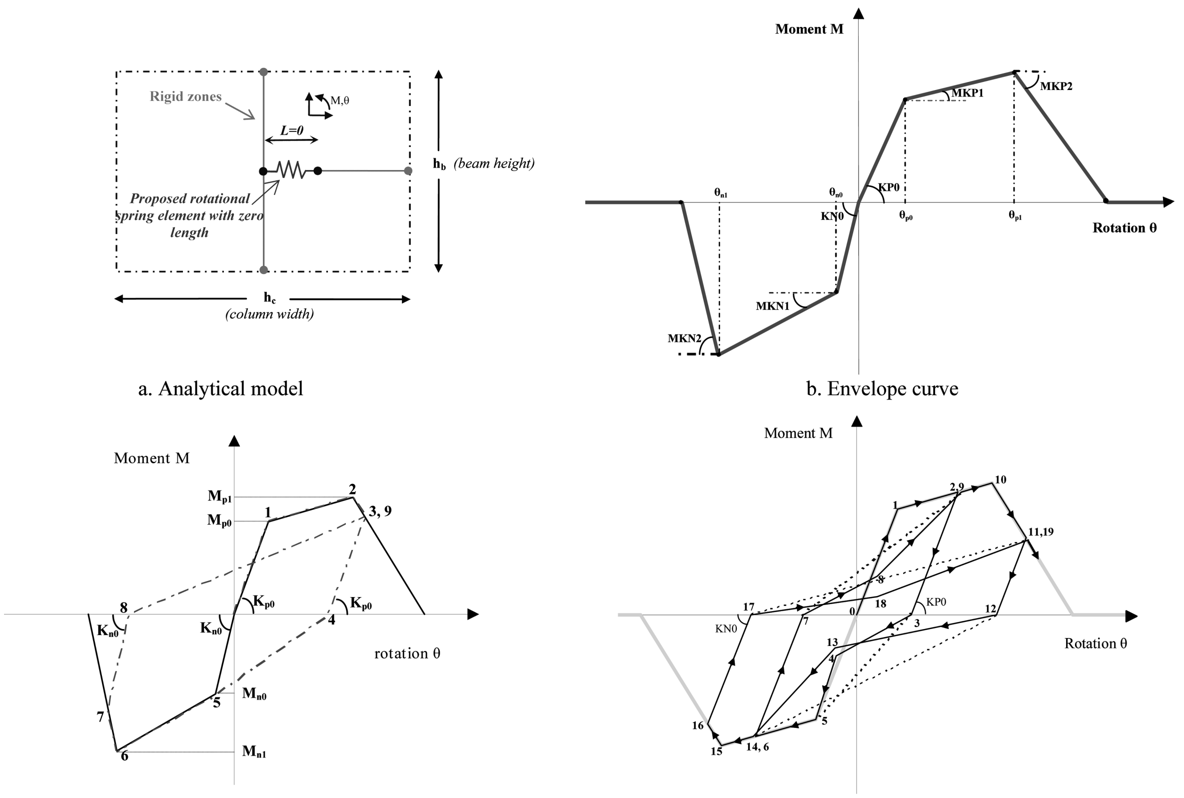


Fig. 2 Response model for the contact element

structure at a point within their 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 (Karayannis and Favvata 2005b). Collisions are simulated using special purpose contact elements that become active when the corresponding nodes come into contact. The response of the contact elements is shown in Fig. 2. 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 infinity stiffness (Fig. 2). The actual 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 (Karayannis and Favvata 2005b). 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. Nevertheless, 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 (Karayannis and Favvata 2005a).

2.2 Modelling the exterior joints local effect

This work is mainly focused on the influence of the exterior beam-column joints possible inelastic response on the pounding between adjacent structures with different total heights and different story



a. Analytical model

b. Envelope curve

c. Response model during a typical hysteretic cycle

d. Hysteretic response of the proposed model including pinching effect

Fig. 3 Joint element - model used for the simulation of the exterior RC beam-column joints heights (Fig. 1). A recently proposed element – model for the simulation of the local inelastic behaviour of the reinforced concrete beam-column joints is used (Favvata 2006, Favvata *et al.* 2008). The enhanced joint element is a spring element with zero length that it is defined by two nodes with the same coordinates (Fig. 3(a)), and it is only influenced by the relative rotational displacements between the nodes. The moment transmitted by the element is the moment transferred from the beam to the column. The entire seismic behaviour of the joint is described by the joint model, and therefore rigid elements are adopted to simulate the portions of the beam and column inside the joint core area (Fig. 3(a)).

The envelope curve is tri-linear with the third branch degrading (Fig. 3(b)). In the response model different envelope curves can be provided for positive and negative deformation of the joint for the case of non-symmetric reinforcement in the adjacent beam. The calibration of the model requires 10 parameters to be defined for the response envelope (KP0, θ_{p0}, MKP1, θ_{p1}, MKP2 for the positive curve, and KN0, θ_{n0}, MKN1, θ_{n1}, MKN2 for the negative curve) and 2 parameters representing positive and negative residual inelastic deformation for the definition of the reloading paths (cyclic response).

The response of the joint element during a typical hysteretic cycle is presented in Fig. 3(c). In this figure, the loading starts in the positive direction with stiffness K_{p0} and continues this way until the "yielding" load (equivalent M_{p0}). Then follows the post-"yielding" curve with stiffness K_{p1} until the load corresponding to the equivalent M_{p1} and later the degrading branch of the positive curve with stiffness K_{p2} until the deformation direction is reversed. Unloading proceeds (points 3-4) with the initial positive stiffness K_{p0} until zero moment. When loading is in the negative direction the stiffness changes and the response segment aims at the negative "yield" point or at the previous unloading point of the negative curve (points 4-5). Then follows the post-"yielding" curve with stiffness K_{n1} until it reaches the load corresponding to the equivalent M_{n1} (points 5-6) and later the degrading branch of the negative curve with stiffness K_{n2} until the load direction is reversed (points 6-7). Unloading proceeds with the initial negative stiffness K_{n0} until zero moment. Loading again in the positive direction aims at the previous unloading point of the positive curve, and the subsequent response proceeds in the same way as described previously.

Moreover, in the hysteretic response model of the joint element pinching can also be included as additional stiffness degradation at the beginning of the reloading path until a predefined rotational deformation. The response of the proposed model including the pinching effect is presented in Fig. 3(d).

For the evaluation of the peak strength of the exterior RC joints in this study the numerical procedure proposed by Favvata *et al.* 2008 is adopted. Based on this procedure the type of the failure mode and therefore the strength of the joint are estimated based on the most critical failure mechanism between: (i) the flexural yield strength of the adjacent beam, (ii) the shear strength of the joint and (iii) the development of maximum bond stress along the horizontal part ℓ_{sp}, of the beam longitudinal reinforcement in the core area of the joint.

The validity and the accuracy of the proposed joint element-model has been demonstrated through comparisons against experimental results of twelve RC exterior beam-column subassemblages under cyclic loading reported in literature by Tsonos 1997, 2002, 2007, Karayannis *et al.* 2006. Results of the joint model verification are presented in Favvata *et al.* 2008, Favvata 2006.

In this study the seismic response of the interior beam-column joints is considered elastic (simulation with rigid zones).

2.3 Simulation of the infill panel

For the simulation of the behaviour of the infill panel the equivalent diagonal strut model is used. The type of element used in this work is a beam-column element that accounts for more accurate definition of the response properties of infilled masonry since it includes degrading branch. Special attention was given in the implementation of this element for the simulation of the infill panel in order to exhibit axial response only. An important problem in modelling the infill panel is the determination of the response characteristics of the diagonal strut model, taking into account the actual conditions of the effective lateral confinement of the masonry by the reinforced concrete frame. In this paper the actual properties of the infill panel have been estimated using experimental results (Karayannis *et al.* 2005).

2.4 Analysis model for structure

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 exceed the pre-defined gap distance (d_g) between the two structures.

The computer program used in this work is the program package ADAPTIC (Izzuddin 1991). For the simulation of the multistory frame structure beams and columns a special quartic element is used. This element involves the use of an elastic formulation capable of accurately modelling a whole member with one element and the automatic refinement of elastic elements into inelastic elements after detection of material inelasticity. Analysis is always started using one elastic quartic element per member, each of which is checked for inelasticity during analysis in predefined regions along its length. If such region becomes inelastic, the elastic element is removed and replaced appropriately by new quartic elastic and cubic inelastic elements (Izzuddin *et al.* 1994, Karayannis *et al.* 1994). In this study four predefined zones (control sections) for inelasticity checks are provided for each elastic quartic element per member. Moreover, 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 quartic elements are employed for this column. Each element is divided in four unequal segments. Thus, there are eight predefined zones for inelasticity checks (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.

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 examined multistory RC structure that suffers the pounding effect the degree of confinement for the external columns was ranged from $K = 1.213$ to 1.309. The confinement degree in the middle part of the internal columns 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. The confinement degree of the column that suffers the hit was equal to 1.292.

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.

3. Description of the examined cases

In the examined cases it is considered that the pounding takes place at points of the deformable height of the columns of the more flexible 8-storey frame structure. Each of the studied cases is examined for three different gap distances between the two structures and is analyzed for the El Centro 1940 seismic excitation with maximum acceleration (α_{max}) scaled to be equal to the design acceleration of the examined structure ($\alpha_{max} = 0.3\ g$). In these interaction cases the total height of the 3-storey structure is greater than the total height of the 3rd floor and less than the total height of the 4th floor of the 8-storey frame and thus the highest contact point of the two structures lies between the levels of the 3rd and the 4th floor of the 8-storey frame. Five positions of the contact point are considered in order to study the influence of the exterior joints local damage on the seismic behaviour of the critical column that suffers the hit. The examined positions are at: 1/6, 1/3, 1/2, 2/3 and 5/6 of the interstory height of the column of 4th floor of the 8-storey frame.

The pounding case between the 8-storey frame and 3-storey height stationary barrier (very stiff structure) is examined considering that the highest contact point is located at the 1/6 of the interstory height of the 4th floor column of the 8-storey frame structure.

The influence of the gap size on the pounding effect is also investigated. Thus, all pounding cases are examined: (a) for structures in contact from the beginning ($d_g = 0$), (b) for initial gap distance between the two structures equal to $d_g = 2.0$ cm and (c) for the case that the structures vibrate independently without pounding effect.

Infill panels: Considering that the masonry infills substantially change the dynamic characteristics of the reinforced concrete structures, an approach to incorporate their influence in the inter-story pounding problem between adjacent structures is attempted. The influence of the masonry infill panels on the seismic response of the multistory structure is investigated considering three types of infilled multistory structures: (a) bare frame structure (for comparison reasons), (b) fully infilled frame and (c) pilotis type frame (soft 1st story).

Ten (10) pounding cases between the 8-storey frame and 3-storey frame-wall structure is examined, considering that the highest contact point is located at the 1/3 of the interstory height of the 4th floor column of the 8-story frame and that the structures are in contact from the beginning

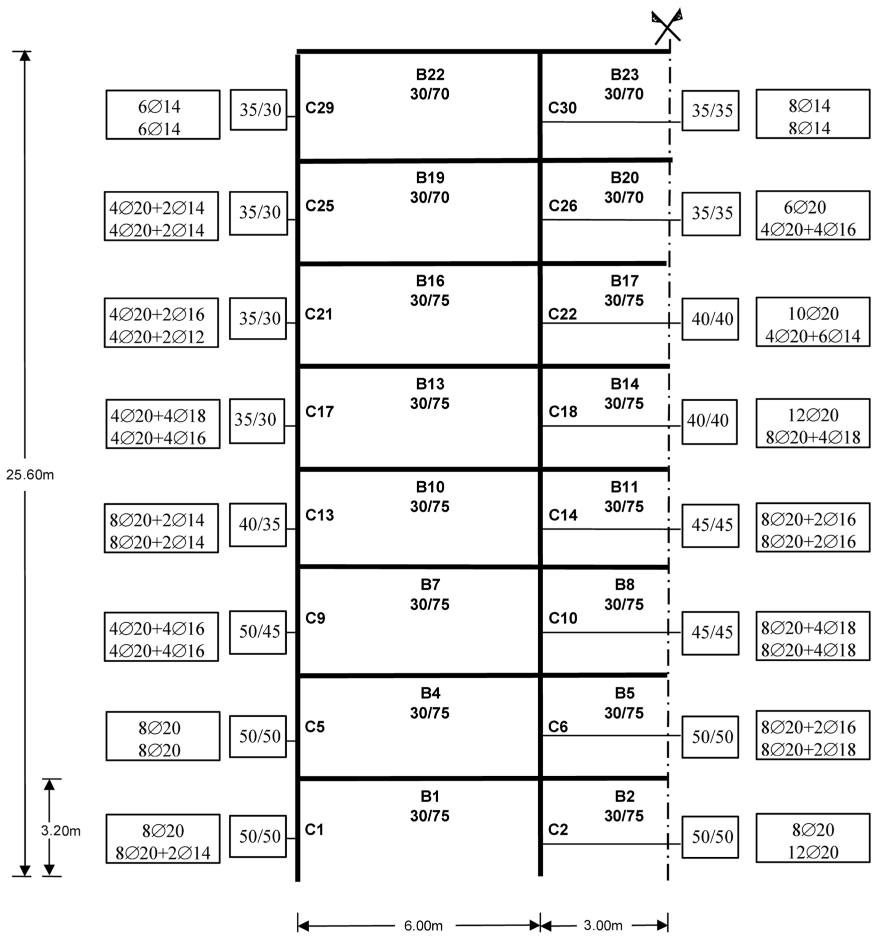
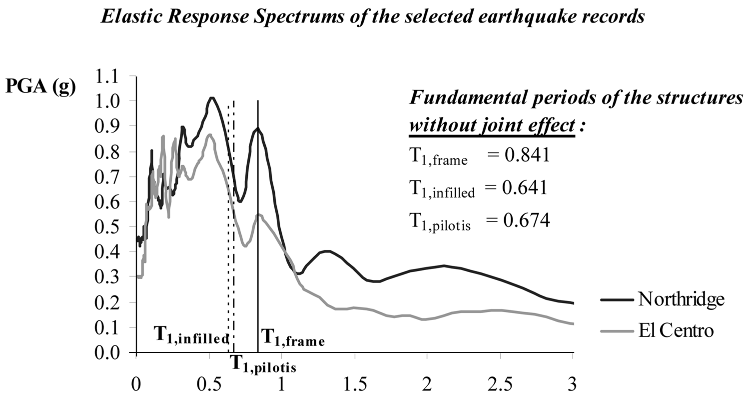


Fig. 4 Structural system and column reinforcement of 8-storey frame designed to EC 2 & 8

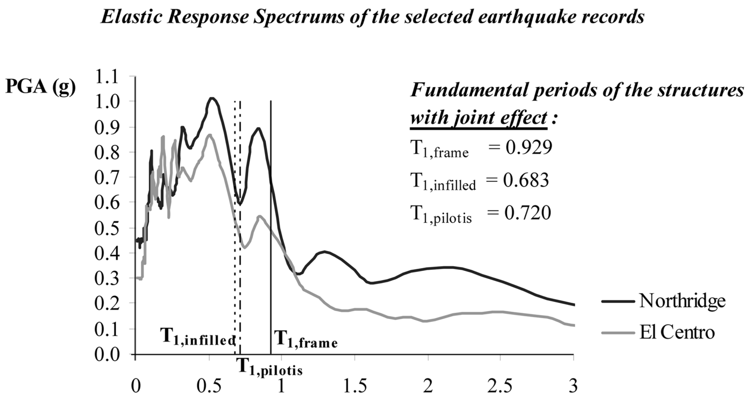
($d_g = 0$). In these cases the structural systems are subjected to a strong seismic action (Northridge, USA 1994 with $\alpha_{max} = 0.45\,g$) and the possible inelastic response of the exterior beam-column joints is studied by including the effect of the infill panels. For comparison reasons the response of the infilled multistory frame structures is also studied without the joints damage effect.

The design of the examined RC buildings is based on the concept that they represent existing buildings without major design deficiencies except for the lack of adequate shear reinforcement in the joints. Structure geometry and reinforcement of the columns of the 8-storey frame are shown in Fig. 4.

The fundamental periods of the examined structures (without and with the proposed joint model) are presented in Fig. 5 along with the elastic response spectrums of the selected earthquake records. The small differences in the period of the structures between the cases with and without the joints local effect are attributed to the fact that linear elastic branch instead of parabolic is used in the joint element model.



(a)



(b)

Fig. 5 Elastic response spectrums of the selected earthquake records and fundamental periods of the examined structures; (a) without joint damage effect, (b) with joint damage effect

4. Seismic response of the critical column

The most important issue in the examined inter-story pounding cases is obviously the 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 the most critical case of interaction between adjacent buildings (Karayannis and Favvata 2005a, b).

In this respect, for the examined interaction cases results concerning the flexural and the shear demands of the critical external column of the 8-storey frame structure that suffers the pounding are presented and compared with the corresponding available flexural and shear capacities. Special attention has been given for the study of the influence of the exterior beam-column joints local damages on the seismic response of this critical column.

4.1 Ductility requirements

The maximum ductility demands developing in the 4th floor column of the 8-storey structure that suffers the hit from the slab of the adjacent shorter and stiffer structure are presented in Tables 1 and 2, for all the examined pounding cases. From these tables it can be observed that the ductility requirements for the critical column are increased when compared with the ones without the

Table 1 Curvature ductility demands of the multistory frame external column that suffers the inter-story pounding (seismic excitation of El Centro 1940 - α_{max} = 0.3 g)

DUCTILITY DEMANDS of the External Column that Suffers the Pounding *seismic excitation of El Centro 1940 α_{max} = 0.3 g*				
Pounding cases between 8-story frame and 3-story structure:				
Pounding at: \ and:	Without joint effect		With joint effect	
	d_g = 0.0 cm	d_g = 2.0 cm	d_g = 0.0 cm	d_g = 2.0 cm
h_A = 1/6h	2.92 (0.85)*	1.77 (0.51)	3.35 (0.97)	1.98 (0.57)
h_A = 1/3h	7.64 (2.21)	2.56 (0.74)	5.69 (1.65)	3.87 (1.12)
h_A = 1/2h	8.74 (2.53)	2.60 (0.75)	5.16 (1.50)	5.74 (1.66)
h_A = 2/3h	5.83 (1.69)	3.95 (1.14)	6.92 (2.01)	2.86 (0.83)
h_A = 5/6h	4.33 (1.26)	1.99 (0.58)	4.65 (1.35)	1.11 (0.32)
Without pounding: Elastic				
Pounding case between 8-story frame and 3-story height rigid barrier structure				
Pounding at: \ and:	Without joint effect		With joint effect	
	d_g = 0.0 cm	d_g = 2.0 cm	d_g = 0.0 cm	d_g = 2.0 cm
h_A = 1/6h	3.97 (1.15)	4.0 (1.16)	3.08 (0.89)	1.33 (0.39)
Without pounding: Elastic				

*Ratio of the ductility demand to the available one.

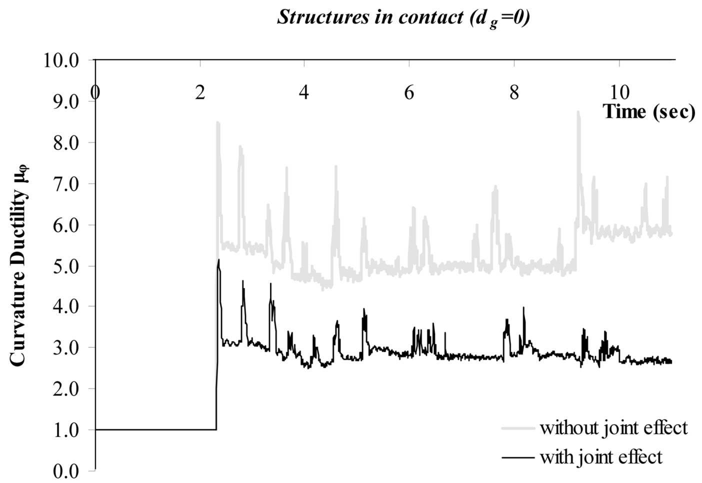
Table 2 Curvature ductility demands of the multistory frame external column that suffers the inter-story pounding (seismic excitation of Northridge 1994 - α_{max} = 0.45 g)

DUCTILITY DEMANDS of the External Column that Suffers the Pounding *seismic excitation of Northridge 1994 - α_{max} = 0.45 g*		
INCLUDING INFILL PANELS: Pounding case between 8-story frame and 3-story structure *Contact point at h_A = 1/3h* and d_g = 0.0 cm		
8-storey structure as: \ and:	Without joint effect	With joint effect
Frame	2.27 (0.66)*	1.16 (0.34)
Infilled Frame	3.62 (1.05)	3.33 (0.97)
Pilotis Frame	2.58 (0.75)	2.44 (0.71)
Without pounding (frame, infilled, pilotis)	Elastic	Elastic

*Ratio of the ductility demand to the available one.

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 initial 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 but in most of the examined cases they appear to be lower than the available ductility values.

Focused on the influence that the possible inelastic response of the exterior beam-column joints may have on the ductility requirements of the column that suffers the impact it can be stated that the damages in the joints area change the flexural response of this column. In this view, the influence of the exterior joints local damages on the time history ductility requirements of the critical column for the interaction case of the 8-storey frame with the 3-storey structure at the point $h_A = 1/2$ of the column height (h), is presented in Fig. 6. From this figure it can be observed that in



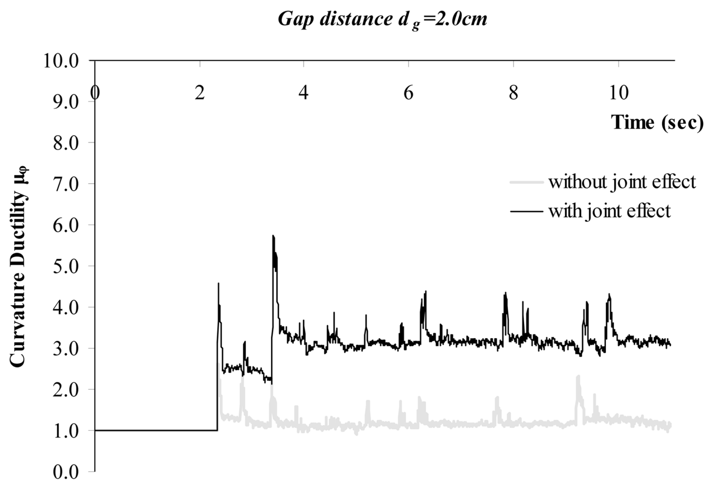


Fig. 6 Results of pounding case between 8-storey frame structure and 3-storey structure at the point $h_A = 1/2h$. Influence of the exterior RC joints local damages on the ductility requirements of the external column of the 4th story that suffers the inter-story pounding

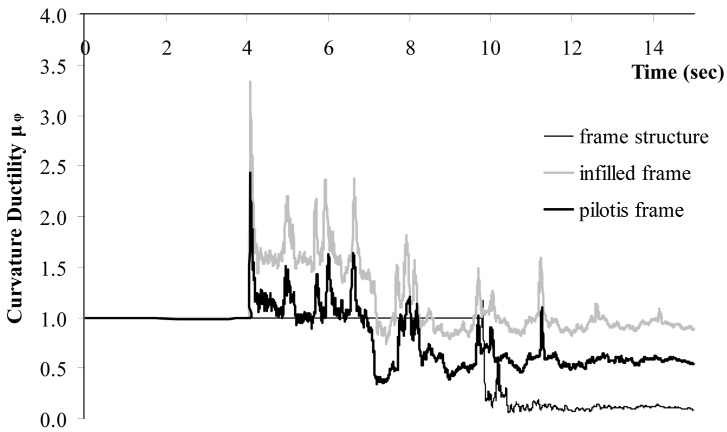


Fig. 7 Influence of the infill panels on the pounding problem of the 8-storey frame structure that includes the exterior joints local effect. Results in terms of time history ductility requirements of the external column of the 4th story that suffers the inter-story pounding (Northridge (USA) 1994 - $\alpha_{max} = 0.45$ g)

the case that the two structures are in contact from the beginning ($d_g = 0$) the exterior joints damages seem to be beneficial for the flexural ductility demands of the column. On the other hand, in case that there is an initial small gap distance between the two structures equal to 2 cm ($d_g = 2$ cm) an increasing ductility demand of the column is observed due to beam-column joints local damage effect.

However, the results of Table 1 indicate that no safe conclusion can be extracted about the influence of the exterior joints seismic behaviour on the maximum ductility requirements of the critical column that suffers the inter-story pounding phenomenon. It is observed that changes on the position that the impact takes place within the deformable height of the column and on the initial gap distance between the adjacent structures induce a different influence of the local inelastic response of the exterior joints on the flexural response of the column.

Finally, in the examined pounding cases that include the response of the masonry infill panels, the influence of the exterior joints local damages on the flexural response of the column that suffers the impact seems to be beneficial (Table 2), although taking into account the infills induces an increase on the ductility demands of the critical column. The influence of the infill panels on the time history ductility requirements of the external column of the 4th story of the 8-storey structure that suffers the inter-story pounding at the point $h_A = 1/3$ of the column height (h) is presented in Fig. 7. It can be observed that during the seismic excitation the ductility requirements of the critical column are generally increased due to the present of the infills.

4.2 Shear requirements of the critical column

The shear demands developing in the critical part of the column of the 8-storey structure that suffers the impact from the adjacent shorter and stiffer structure are presented in Tables 3 and 4, for all the examined pounding cases. These tables present the increase of the developing maximum shear forces due to pounding effect. Furthermore, the number of time steps during the analysis that the shear forces of the critical part of the column exceed its shear strength is also given in Tables 3 and 4.

Table 3 Shear demands of the multistory frame external column that suffers the inter-story pounding (seismic excitation of El Centro 1940 - $\alpha_{\max}$ = 0.3 g)

SHEAR DEMANDS of the External Column that Suffers the Pounding *seismic excitation of El Centro 1940 - α_{max} = 0.3 g*				
Pounding cases between 8-story frame and 3-story structure:				
Pounding at: \ and:	Without joint effect		With joint effect	
	d_g = 0.0 cm	d_g = 2.0 cm	d_g = 0.0 cm	d_g = 2.0 cm
h_A = 1/6h	5.43[⊕] (401)[⊗]	4.87 (175)	4.66 (308)	3.63 (211)
h_A = 1/3h	2.88 (250)	2.72 (133)	2.63 (213)	2.36 (155)
h_A = 1/2h	1.75 (150)	1.57 (70)	1.39 (61)	1.49 (40)
h_A = 2/3h	2.14 (264)	2.41 (165)	1.92 (252)	1.87 (137)
h_A = 5/6h	3.58 (334)	3.59 (173)	2.52 (297)	2.81(142)
Without pounding: Elastic response				

Pounding case between 8-story frame and 3-story rigid structure *Contact point at h_A = 1/6h* and d_g = 0.0 cm	
Without joint effect	With joint effect
4.67 (710)	3.99 (359)
Without pounding: Elastic response	

[⊕]Ratio of the maximum shear demand to the available shear strength.
[⊗]Indicates the number of times steps during the analysis that the shear exceeds the available shear strength.

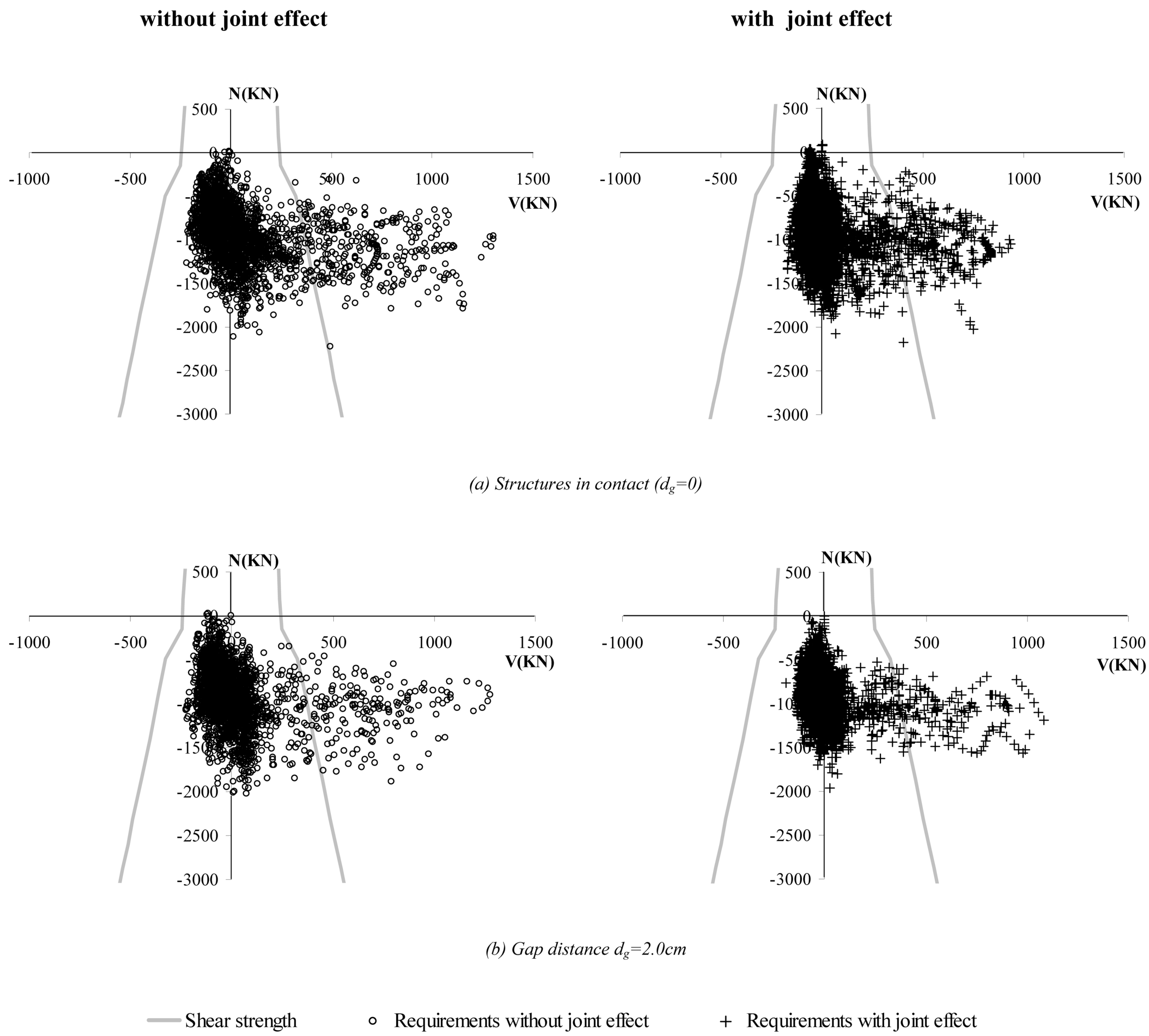
Table 4 Shear demands of the multistory frame external column that suffers the inter-story pounding (seismic excitation of Northridge 1994 - $\alpha_{\max}$ = 0.45 g)

SHEAR DEMANDS of the External Column that Suffers the Pounding *seismic excitation of Northridge 1994 - α_{max} = 0.45 g*		
INCLUDING INFILL PANELS: Pounding case between 8-story frame and 3-story structure *Contact point at h_A = 1/3h and d_g = 0.0 cm*		
8-storey structure as: \ and:	Without joint effect	With joint effect
Frame	1.95 (31)	1.66 (28)
Infilled Frame	2.54 (49)	2.50 (92)
Pilotis Frame	2.48 (78)	2.41 (35)
Without pounding (frame, infilled, pilotis): Elastic response		

[⊕]Ratio of the maximum shear demand to the available shear strength.
[⊗]Indicates the number of times steps during the analysis that the shear exceeds the available shear strength.

It can be observed that the inter-story pounding effect induces a critical shear state for the seismic performance of the column since in all the examined cases the developing shear forces exceed the capacity for shear strength of the member many times during the excitation (Tables 3, 4). Moreover, the pounding cases where the highest contact points are considered at the 1/6 and 5/6 of the interstory height of the column of the 4th floor are proved to be the most critical interaction cases for the safety of the 8-storey structure (Table 3). More intense seems to be the inter-story pounding phenomenon for the column that suffers the impact in the interaction cases between the 8-storey frame and the 3-storey height stationary barrier (very stiff structure) (Table 3).

For the interstory pounding case of the 8-storey frame with the 3-storey frame-wall structure at the point $h_A = 5/6$ of the column height (h), the influence of the exterior beam-column joints damages is presented in Fig. 8 for the cases that the two structures are (i) in contact from the beginning ($d_g = 0$), and (ii) there is an initial small gap distance between the adjacent structural systems equal to 2 cm ($d_g = 2$ cm). In this figure the points represent the pairs of the developing shear force, V, and the axial force, N, at every step of the seismic analysis, whereas the lateral solid lines show the available capacity of the reinforced concrete element for the combination of shear



(a) Structures in contact (d_g=0)

(b) Gap distance d_g=2.0cm

Fig. 8 Results of pounding cases between 8-storey frame structure and 3-storey structure at the point $h_A = 5/6h$. Shear forces developing in critical column of the 4th story of the 8-storey frame structure and available strength. Influence of the exterior RC joints local damage

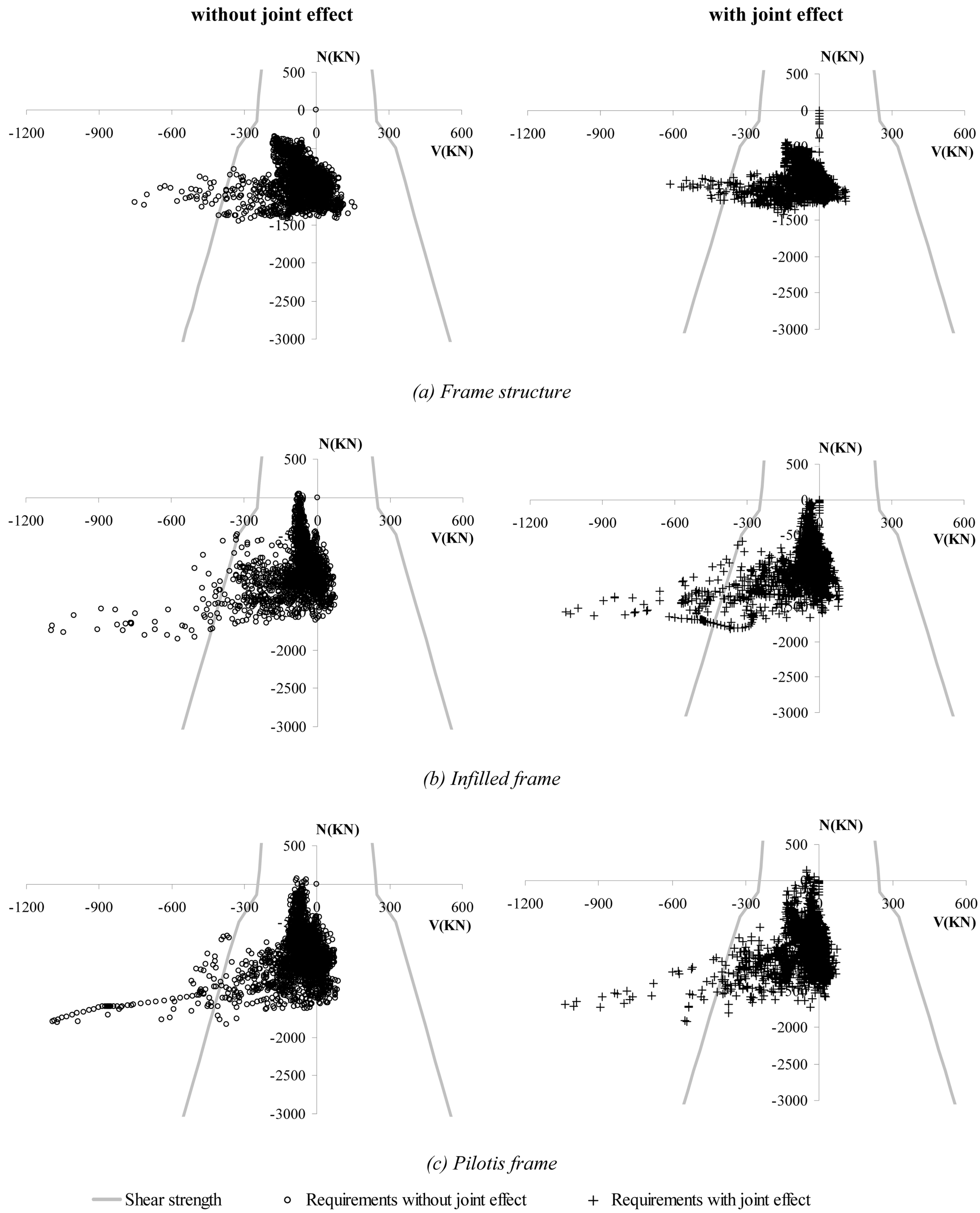


Fig. 9 Influence of the infill panels on the pounding problem of the 8-storey frame structure. Developing shear forces of the external column of the 4th story that suffers the inter-story pounding (Northridge (USA) 1994 - $\alpha_{\max} = 0.45$ g)

versus axial force (EC2 & 8). This way a direct comparison of the developing shear force at all the steps of the analysis with the available shear strength can be obtained.

It can be observed that the damages of the exterior joints lead to a decrease of the requirements for shear strength of the column. Based also on the results of Table 3 it can be deduced that the exterior joints local damages seem to decrease the magnitude of the developing shear forces of the critical column that suffers the hit when compared to the corresponding values developed without considering the joints local effect.

Similarly, in the cases that masonry infills are included in the analysis model of the 8-storey structure, it can be observed that the maximum shear demands of the column that suffers the hit are decreased due to the exterior joints local damage effect, too (Table 4).

It is noted that in the examined cases the pounding effect seems to be more intense for the critical column when masonry infills are incorporated in the analyses. Fig. 9 presents the developing shear forces of the column of the 8-storey structure that suffers the hit at the point $h_A = 1/3h$ from the slab of the 3-storey structure and the two structures are in contact from the beginning ($d_g = 0$) for the cases that the multistory structure is modeled as: (a) frame structure without infills, (b) fully infilled frame structure, and (c) infills are considered in the 2nd-8th stories and thus soft 1st story (pilotis building). In this figure the developing shear demands of the column are compared with its corresponding capacity value, while results for both cases: (i) without joints effect, and (ii) with exterior joints local effect, are also presented. It is observed that the influence of the infill panels on the seismic performance of the column that suffer the hit has as a result a considerable increase of the shear demands when compared to the corresponding values that are developed in case that the multistory frame is studied without considering the infills.

5. Local inelastic response of the exterior joints

This study is also aiming in the investigation of the influence of the inter-story pounding on the local response of the exterior beam-column joints of the 8-storey frame structure. Results in terms of maximum rotational deformation requirements of all the exterior joints at the pounding side of the multistory structure are presented and compared with the capacity levels of deformation at the beginning of the joint first cracking (θ_{p0}) and at the beginning of the joint stiffness degradation branch (deformation θ_{p1}, - Fig. 3(b)). Thus, a direct comparison of the joint capacity with the joint's rotational requirements can be presented, and the magnitude of the deterioration of the exterior joints can be identified.

In all the examined cases, the seismic behaviour of the exterior joints of the upper floor levels of the 8-storey frame (without infills) is proved to be critical for the safety of the structure. Fig. 10 shows the seismic performance of the exterior joints of the tall building that suffers the inter-story pounding from the 3-storey frame-wall structure and for the cases that the highest contact point is located at: (a) the 1/3 of the interstory height of the 4th floor column of the 8-storey frame, (b) the 2/3 of the interstory height of the 4th floor column of the 8-storey frame, and (c) the 1/2 of the interstory height of the 4th floor column of the 8-storey frame. In this figure the rotational demands of the exterior joints are presented for the cases that the structures are: (i) in contact from the beginning ($d_g = 0$), (ii) there is a small initial gap distance between the structural systems equal to 2 cm ($d_g = 2.0$ cm), and (iii) without the interaction pounding effect.

It is observed that the local response of the exterior beam-column joints from 5th to 7th floor is

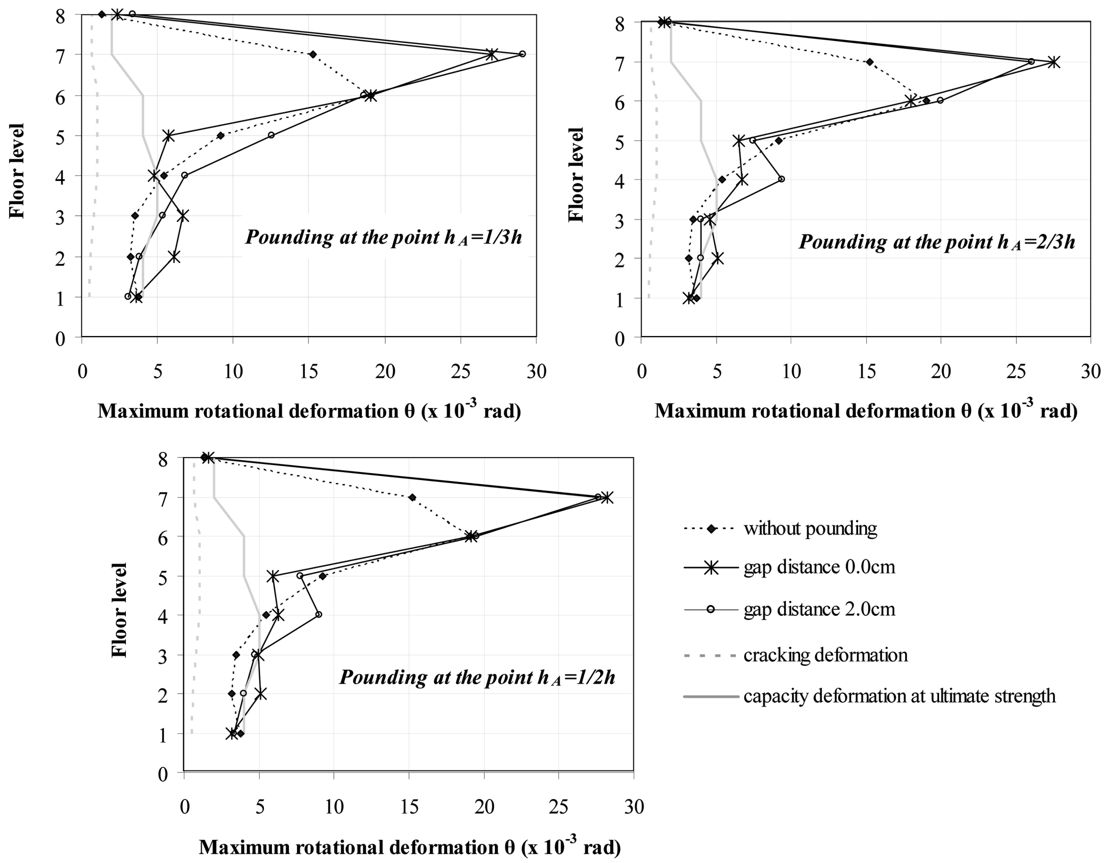


Fig. 10 Influence of the inter-story pounding on the maximum rotational deformation requirements of the exterior RC joints (at the pounding side)

characterized by serious damages due to high rotational demands (Fig. 10). Moreover, the inter-story pounding effect induces an important increase on the deformation requirements especially on the exterior joint of the 7th floor of the 8-storey frame structure.

Also, critical seems to be the influence of the pounding effect on the seismic response of the exterior joints below the contact point between the adjacent structures. It is observed that the interaction between the structural systems causes damages in the beam-column joint of the 4th floor of the multistory frame structure (and in some cases in the joints of the 2nd and 3rd floors) when compared to the corresponding responses without the pounding effect (Fig. 10).

The seismic performance of the exterior beam-column joints when masonry infills are included in the analysis model of the tall building is shown in Fig. 11. The results of this figure are for the pounding case between the 8-storey frame and the 3-storey frame-wall structure considering that the highest contact point is located at the 1/3 of the interstory height of the 4th floor column of the 8-storey frame and that the structures are in contact from the beginning ($d_g = 0$). The influence of the infill panels on the developing maximum demands for deformation of the exterior beam-column joints is presented for the cases that the 8-storey structure is being modeled as: (a) frame structure without infills, (b) fully infilled structure, and (c) pilotis type building. The results mainly indicate that the overall developing maximum rotational requirements of the exterior joints are significantly

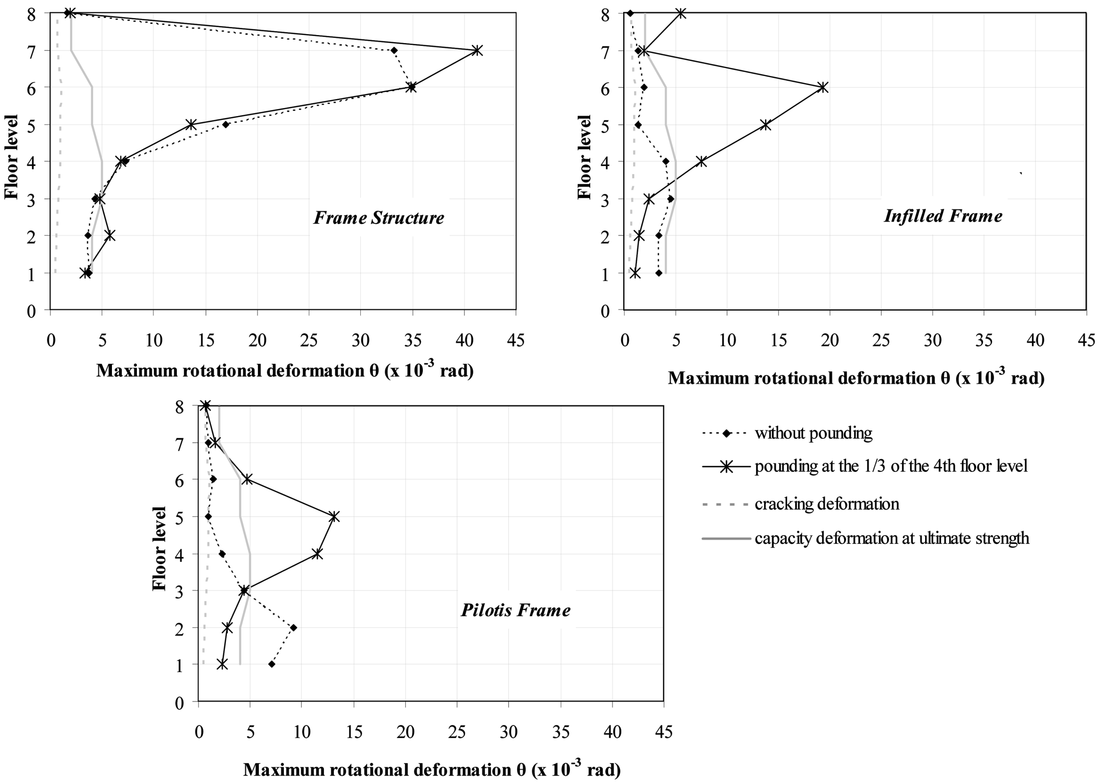


Fig. 11 Influence of the infill panels on the maximum rotational deformation requirements of the exterior joints (Northridge (USA) 1994 – $\alpha_{\max}$ = 0.45 g)

reduced when the masonry infills are considered in the analyses.

In Fig. 12 results of the hysteretic response of the exterior joint of the 4th floor level of the tall structures (without and with infill panels) in the area of the impact are presented. These results clearly indicate that the interaction pounding between the adjacent structures significantly changes the local inelastic response of the exterior joint in comparison to the corresponding response of the case that the examined structures are vibrating independently.

In Fig. 12 it can be observed that due to the pounding effect the maximum demand for rotational deformation of the exterior joint of the 4th floor level of the 8-story frame structure is almost equal with the corresponding maximum requirement developed in the core area of the joint when the structure has infill panels.

More intense seems to be the influence of the pounding problem on the seismic performance of the exterior joint of the 4th floor level of the pilotis type structure (Fig. 12(c)). In this case severe damages inside the joint area have been observed. Based on the hysteretic response of the joint it is noted that the joint in the case without the pounding exhibited an almost elastic behaviour whereas in the case with pounding increased inelastic maximum deformation demands have been developed (Fig. 12(c)).

It is also worthy to be noted that in case of analyzing the structure as pilotis (soft 1st story) the response of the exterior beam-column joints at the lower floor levels (1st and 2nd) of the 8-storey structure that vibrating independently (without pounding) is proved to be critical for the safety of

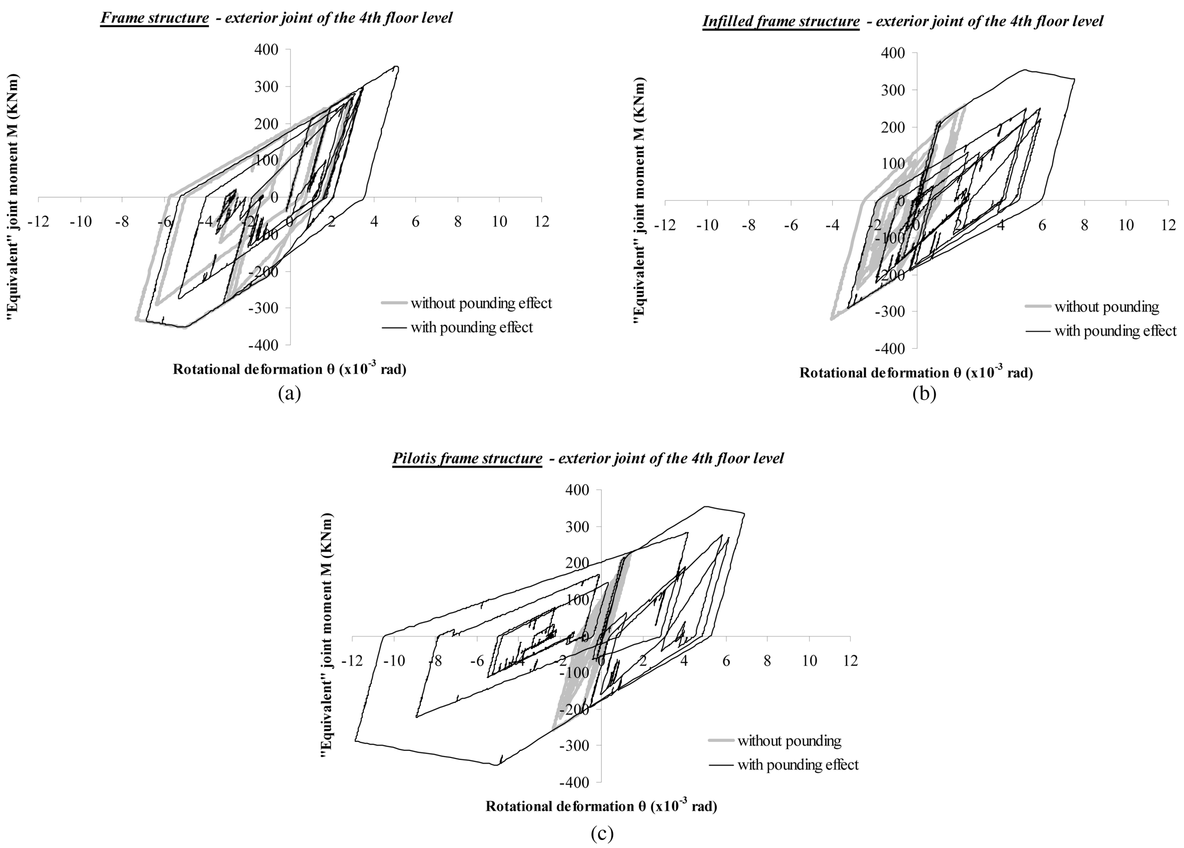


Fig. 12 Influence of the inter-story pounding on the hysteretic response of the exterior joint of the 4th floor level of the tall structures in the area of the impact

the structure since in that case the maximum developing rotational demands of the joints at these floor levels exceed the corresponding capacity deformation at ultimate strength (Fig. 11).

6. Conclusions

In this work the exterior RC joints local inelastic response is included as key parameter for the study of the inter-story pounding problem between adjacent structures. Although several parameters that influence the structural pounding have been studied sofar the role of the joints local damage has not been yet investigated in the literature as key parameter for the pounding problem. Moreover, the influence of the infill panels as an additional for the beam-column joint local effect parameter on the investigation of the seismic performance of multistory frame structures that suffer the pounding from an adjacent shorter and stiffer structure is examined.

The main remarks and conclusions of this investigation can be summarized as follows:

(i) The column that suffers the inter-story pounding effect 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.

(ii) It was observed in the examined cases that the shear requirements of the critical column that suffers the hit are decreased when in the analysis model of the multistory structure the possible inelastic response of the exterior beam-column joints is considered.

(iii) No safe conclusion can be extracted about the influence of the exterior joints damages on the maximum demands for ductility of the critical column that suffers the hit. It has been observed that the results concerning the joints local damage effect on the flexural response of the column were influenced by changes on the position that the impact takes place within the deformable height of the column and on the initial gap distance between the adjacent structures.

(iv) The influence of the infill panels on the seismic performance of the column that suffers the hit led in all cases to an increase of the demands for shear and ductility of the critical column, when compared to the corresponding values that are developed in the cases that the multistory frame is studied without considering the infills.

(v) Similarly critical is proved to be the influence of the pounding on the seismic response of the exterior beam-column joints at the floor levels where the interaction takes place. In any case, the interaction between the structural systems causes damages in the joints of the multistory frame structure when compared to the corresponding responses without the pounding effect.

(vi) The developing maximum rotational requirements of the exterior joints are significantly reduced in cases that masonry infills are taken into account. This is not the case of pilotis type structure(soft story) where due to the pounding effect high level of rotational deformations demands were concentrated in the core area of the exterior joint at the contact floor level (4th floor).

(vii) The case of pilotis type structure was proved to be the most critical one for the seismic performance of the column that suffers the hit and for the exterior joint in the area of the impact in terms of shear and inelastic deformation demands, respectively.

(viii) Finally, the existence of a gap between the adjacent structures decreases the high ductility demands of the columns that suffer the hit. It is though not equally effective for the amelioration of the shear demands of columns and exterior beam-column joints at the impact levels.

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