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Influence of pinching effect of exterior joints on the seismic behavior of RC frames

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Abstract. Nonlinear dynamic analyses are carried out to investigate the influence of the pinching hysteretic response of the exterior RC beam-column joints on the seismic behavior of multistory RC frame structures. The effect of the pinching on the local and global mechanisms of an 8-storey bare frame and an 8-storey pilotis type frame structure is evaluated. Further, an experimental data bank extracted from literature is used to acquire experimental experience of the range of the real levels that have to be considered for the pinching effect on the hysteretic response of the joints. Thus, three different cases for the hysteretic response of the joints are considered: (a) joints with strength and stiffness degradation characteristics but without pinching effect, (b) joints with strength degradation, stiffness degradation and low pinching effect and (c) joints with strength degradation, stiffness degradation and high pinching effect. For the simulation of the beam-column joints a special-purpose rotational spring element that incorporates the examined hysteretic options developed by the authors and implemented in a well-known nonlinear dynamic analysis program is employed for the analysis of the structural systems. The results of this study indicate that the effect of pinching on the local and global responses of the examined cases is not really significant at early stages of the seismic loading and especially in the cases when strength degradation in the core of exterior joint has occurred. Nevertheless in the cases when strength degradation does not occur in the joints the pinching may increase the demands for ductility and become critical for the columns at the base floor of the frame structures. Finally, as it was expected the ability for energy absorption was reduced due to pinching effect.

Keywords: hysteretic response; pinching; strength degradation; stiffness degradation; RC beam-column joints; energy dissipation; multistory RC frame structures; seismic responses

1. Introduction

The inelastic response and the stiffness deterioration of reinforced concrete (RC) beam-column joints have been recognized as important parameters that have to be considered carefully in the seismic analysis of structures especially in cases of old buildings. Existing building stock that constitutes the main part of the center of the big cities in Europe are buildings with very small amount of shear reinforcement or no stirrups at all in the joint areas even in high seismic zones. It is stressed that the local damage of the exterior joints has been repeatedly identified as leading

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cause of failure and eventual collapse of RC buildings (Paulay and Priestley 1992). Literature provides significant experimental and analytical research that focused on the study of the seismic performance of RC beam-column connections and further on the identification of their failure mechanisms. An overview of analytical models that have been reported in the literature the last two decades for the evaluation of the shear strength of the exterior RC joints are reported by Lima *et al.* (2012).

The most important types of failure mode that are frequently observed in the case of exterior RC joints are mainly two: (a) shear failure due to inadequate shear reinforcement in the joint and (b) bond failure due to insufficient anchorage. A significant parameter that characterizes the overall seismic performance of the joints is the type of the developing hysteretic behavior which it further relates to the failure mode. The stiffness degradation and the pinching are the two characteristics which additional to the strength degradation define the hysteresis type of the joints. Several hysteresis models ranging from simple to more complex models have been developed to capture the actual degrading response of the structural members subjected to cyclic loading (e.g. Takeda *et al.* 1970, Kunnath *et al.* 1997, Ibarra *et al.* 2005, Erberik *et al.* 2012). In this view, Kwak *et al.* (2004) proposed a well refined model for the nonlinear dynamic analysis of RC structures, which was calibrated against experimental results.

In 2009, Huang and Foutch investigated the effect of the hysteresis type on the global seismic responses of multistory steel moment frame structures. For this purpose the authors employed a rotational spring at the ends of the beam element to model the inelastic behavior of the beams taken into account three different rules of the hysteretic behavior model.

Also, Sharma *et al.* (2011) presented an analytical model for the inelastic shear behavior of the exterior RC joints. The envelope curve that describes the behavior of the joint includes a degrading branch (strength degradation) and it is evaluated in terms of principal tensile stress vs. shear deformation.

An approach to estimate the hysteretic behavior of the RC beam - column joints with limited shear reinforcement was also presented by Sengupta and Li (2013). In this study the authors adjusted the Bouc-Wen hysteresis model to represent the actual characteristics of the degrading response of the joints.

Several approaches focused on the simulation of RC joints with various degrees of accuracy have been presented 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). However, such models are difficult to be employed in the seismic analysis of an entire multistory RC frame structure.

There are limited studies reported in the literature that include the influence of the damage and failure of the joints on the seismic response of multistory RC frames. In these studies a simple rotational spring element is typically adopted for the simulation of the local effect of the joints (Ghobarah and Biddah 1999, Calvi *et al.* 2002). Pampanin *et al.* (2003) investigated the effects of the joint damage on the overall seismic response of existing frame systems designed for gravity-load only.

Recently a special purpose model for the simulation of the RC beam-column joints local response has been developed. It has been implemented in a general purpose program (ADAPTIC, Izzuddin 1991) for non-linear static and dynamic analysis of structures as a spring joint element (Favvata PhD 2006, Favvata *et al.* 2008). This joint element-model describes the basic characteristics of the RC joints hysteretic response: strength degradation, stiffness degradation and pinching. In 2009, Favvata *et al.* used this joint element in order to include as a key parameter the local damage of the exterior RC joints in the study of the inter-story pounding problem between adjacent RC structures. Thereafter Karayannis *et al.* (2011) using the same joint element (Favvata *et al.* 2008) investigated the way that the strength and stiffness degradation of RC beam-column joints affect the seismic response of multistorey RC frame structures. Nevertheless, in both of these studies the hysteretic response of the joints is considered without the pinching effect.

The aim of this study is to investigate the influence of the pinching response of the exterior RC joints with reduced capacity (without seismic detailing) on the seismic performance of multistory RC frame structures. Dynamic step by step seismic analyses for three different strong motion natural excitations are performed and special purpose elements are employed for the needs of this study. Results showing the influence of pinching on the local and the global mechanisms of an 8-storey bare frame and an 8-storey pilotis type frame structure are presented in terms of hysteretic responses of the joints, joints capacities for energy absorption, maximum accumulated energies, rotational requirements of the joints responses, curvature ductility requirements of columns, requirements for top displacement of the structures and interstory drifts.

2. Response model of RC beam-column joints

2.1 Main considerations – envelope curve

Recently an efficient and accurate model for the simulation of the local damage and deterioration of exterior RC beam-column joints that can be readily used in the analysis of multistory RC frame structures has been reported by Favvata *et al.* (2008). The effectiveness of this joint element model has been demonstrated through comparisons with experimental data and other models reported in literature. This model has been successfully incorporated in the well established nonlinear dynamic structural analysis program ADAPTIC (Izzuddin 1991) providing this way an integrated tool for the investigation of the influence of the nonlinear behavior of low capacity joints on the actual seismic response of multistory RC structures.

This improved joint element model can be considered as a spring element with zero length, it is defined by two nodes with the same coordinates, and it is only influenced by the relative rotational displacements between these nodes. The moment transmitted by the element is the moment transferred from the beam to the column. The entire local behavior of the joint is described by the proposed joint model, and therefore rigid elements are adopted to simulate the portions of the beam and the columns inside the joint core area (Favvata *et al.* 2008).

The model has a trilinear envelope curve with the third branch degrading. In this way, a clearer understanding for the properties associated with the curves is established, and a better convergence of the algorithm is achieved. In the formulation of the proposed 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. Calibration of the model requires 10 parameters to be defined for the response envelope (KP0, θ_{yp}, MKP1, θ_{p1}, MKP2 for the positive curve, and KN0, θ_{yn}, MKN1, θ_{n1}, MKN2 for the negative curve, see Fig. 1) and 2 parameters representing positive and negative residual inelastic deformation for the definition of the reloading paths (cyclic response).

2.2 Pinching response

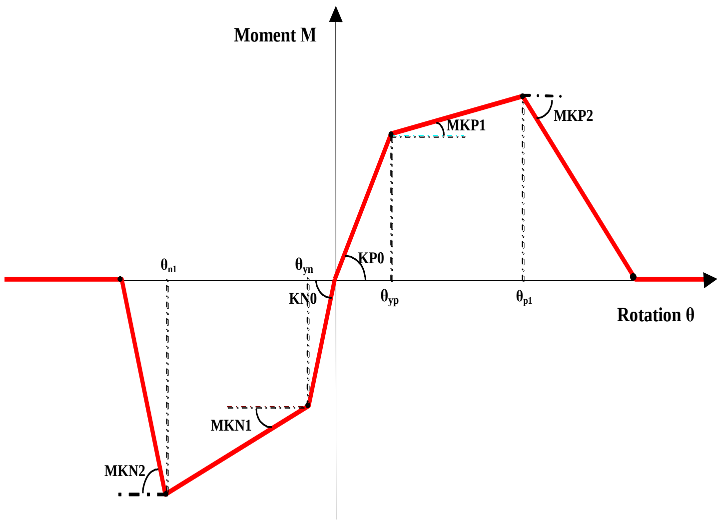
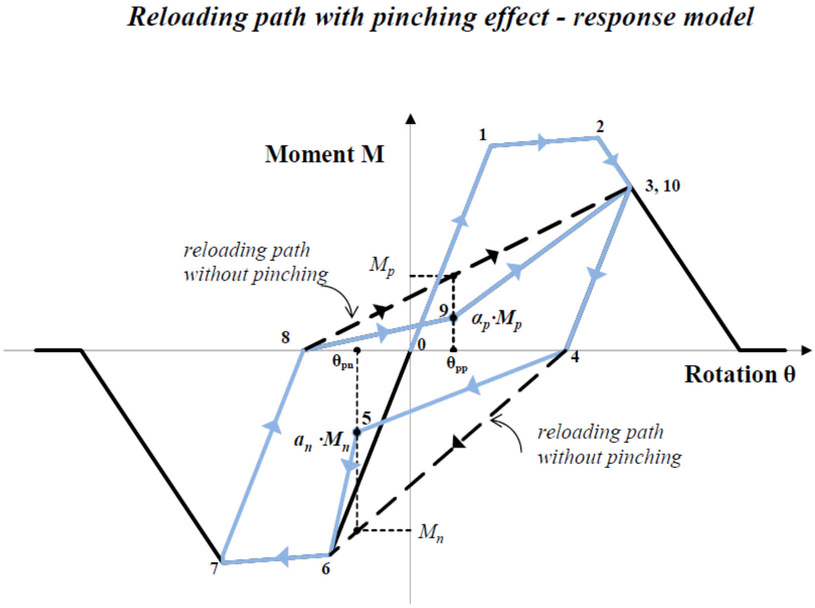


Fig. 1 Envelope curve of the behavior model of the exterior RC beam-column joints with reduced capacity

An important characteristic of the cyclic response of the reinforced concrete beam-column joints is the reduced capacity for energy dissipation which is depicted in the hysteresis loops through pinching. Moreover, the behavior of the joints under cyclic loading is also characterized by strength and stiffness degradation. In this aspect, the basic rules for the unloading-reloading paths of the response model are: (a) unloading stiffness is equal to the initial stiffness, and (b) reloading path follows the direction that aims at the previous unloading point of the basic response curve.

The response of the proposed joint element model including the pinching effect is presented in Fig. 2. In this figure, the loading starts in the positive direction with stiffness K_{p0} and continues this way until the "yielding" load (see also Fig. 1). It follows the post-"yielding" curve with stiffness MK_{p1} and then follows the degrading branch of the positive curve with stiffness MK_{p2} until the deformation direction is reversed (points 0-1-2-3). Unloading proceeds (points 3-4) with the initial positive stiffness K_{p0} until zero moment. In order to include the pinching effect additional stiffness degradation can be predefined at the beginning of the reloading path until a predefined rotational deformation (positive deformation θ_{pp} and negative deformation θ_{pn}). Thus, when loading is in the negative direction the stiffness changes and the response segment aims at the predefined negative rotational deformation θ_{pn} and then at the negative "yield" point or at the previous unloading point of the negative curve (points 4-5-6). Right after loading the post-"yielding" curve follows with stiffness MK_{n1} (points 6-7) until the load direction is reversed. Unloading proceeds with the initial negative stiffness K_{n0} until zero moment (points 7-8). Reloading in the positive direction aims at the predefined positive rotational deformation θ_{pp} and then at the previous unloading point of the positive curve (points 8-9-10). In each new cyclic loading, the stiffness at the beginning of the reloading path is additionally reduced until the predefined rotational deformation. The load capacities in the predefined rotational deformations are also reduced in comparison to the previous cyclic load through the pinching factors α_p and α_n for positive and negative loading directions, respectively. The behavior of the joint element during a typical hysteretic cycle without pinching is also presented in Fig. 2.



1st full loading cycle with pinching effect: points 0-1-2-3-4-5-6-7-8-9-10 (3)
1st full loading cycle without pinching effect: points 0-1-2-3-4-6-7-8-9-10 (3)

Important points 5 and 9 are determined through the pinching factors α_p and α_n for positive and negative loading part, respectively.
Determination of pinching factors α_p and α_n using experimental data (see Table 1)

Fig. 2 Hysteresis response model that includes strength degradation stiffness degradation and pinching

3. Initial stiffness of the joint

The efficiency of the adopted rotational joint element depends on the accuracy of the evaluation of certain model parameters: initial stiffness, ultimate strength of the joint and the characteristics of the expected hysteretic response (strength degradation, stiffness degradation and pinching parameters).

Parametric investigation was performed in order to obtain a reliable and accurate approach for the estimation of the initial elastic behavior of the RC exterior joint and therefore the elastic stiffness parameter for the joint rotational element model. The need for accurate determination of the real initial stiffness of the exterior RC joints has also been emphasized in a recent work by Park and Mosalam (2009). In this view, Kim and LaFave (2007) also proposed high stiffness to be assigned in the joint capacity curve up to the initiation of diagonal cracking.

Thus, taking into account that the entire local response of the exterior joint is described by the rotational element the portions of the beam and column inside the joint core area are simulated using rigid elements. Nevertheless the initial stiffness of the joint has to be rather the same whether the analysis includes rotational element model or not.

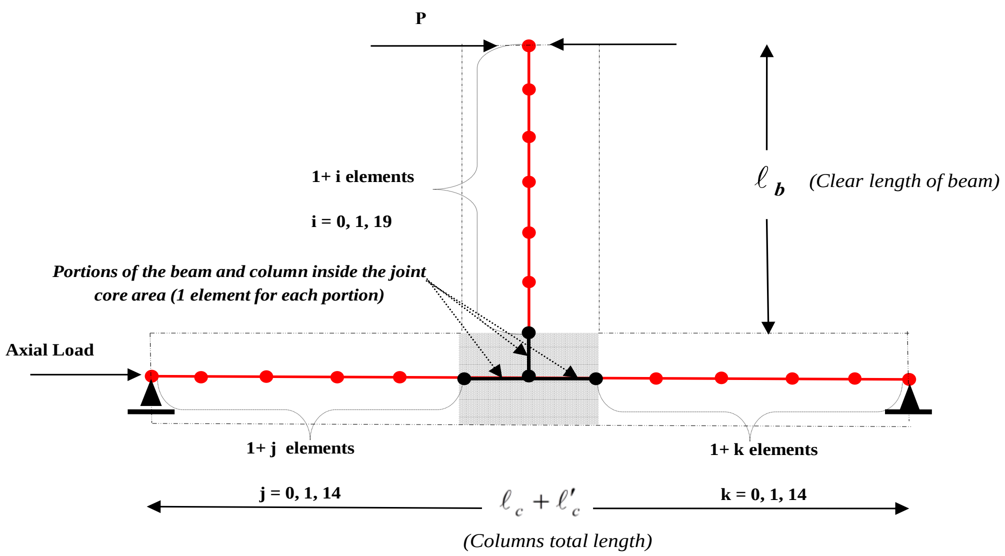
The investigation included eleven different types of analytical simulation for the RC exterior beam-column joints and the comparison of the elastic analyses with experimental data results (see Favvata 2006). In Fig. 3(a) the adopted analytical model of the RC exterior beam-column subassemblages under cyclic loading is presented. More details about the overall analyses performed can be found in Favvata (2006). The results of these elastic analyses and the

Table 1 Pinching factors α_p and α_n using experimental data

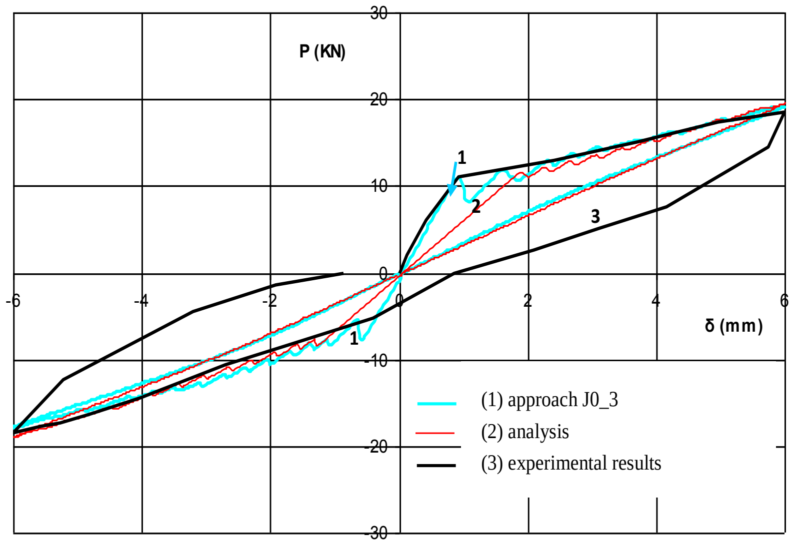
Reference	Specimen	$v_c = \frac{N}{f_c A_c}$ (1)	$\frac{\sum M_{Rc}}{M_{Rb}}$	$\frac{h_b}{h_c}$ (2)	Reinforcement inside the joint	f_c [Mpa]	f_y[Mpa] (3)	α_p (4)	α_n (4)
Pantelides *et al.* 2002	Unit 1	0.10	1.87	1.0	-	33.1	464.15	0.6	0.3
Pantelides *et al.*2002	Unit 4	0.25	2.09	1.0	-	31.6	464.15	0.33	0.33
Pantelides *et al.* 2002	Unit 5	0.10	1.85	1.0	-	31.7	464.15	0.45	0.45
Pantelides *et al.* 2002	Unit 6	0.25	2.04	1.0	-	31	464.15	0.4	0.6
Pampanin *et al.*2002	L1	0.0	0.28	1.65	-	23.9	365.75	0.25	0.25
Tsonos 2002	O1	0.25	0.92	1.5	-	16	485	0.24	0.24
Tsonos 2002	O2	0.25	0.92	1.5	-	16	485	0.21	0.21
Tsonos 2007	L1	1.29	0.94	1.5	-	34	527.5	0.30	0.30
Karayannis *et al.*1998	J0	0.1	2.0	1.0	-	20.78	525	0.5	0.5
Karayannis *et al.*2003	J0	0.05	1.70	1.5	-	32.8	584	0.625	0.625
Karayannis *et al.*2003	A1, A2	0.05	1.76	1.5	-	36.4	520	0.62	0.62
Karayannis *et al.*2008	A0	0.05	1.66	1.5	-	31.6	580	0.5	0.65
Karayannis *et al.*2008	B0	0.05	1.04	1.0	-	31.6	580	0.23	0.3
Karayannis *et al.*2008	A1-R	0.05	1.66	1.5	1Ø8	31.6	580	0.5	0.4
Karayannis *et al.*2008	A2-R	0.05	1.66	1.5	2Ø8	31.6	580	0.5	0.43
Karayannis *et al.*2008	B1	0.05	1.04	1.0	1Ø8	31.6	580	0.35	0.36
Tsonos *et al.* 1992	S2	0.16	1.38	1.5	3Ø8	26	496.84	0.2	0.2
Hakuto *et al.*2000	O6	0		1.08	1-R6 hoop	34	308	0.35	0.35
Hakuto *et al.*2000	O7	0		1.08	1-R6 hoop	31	398	0.45	0.45
Ehsani & Wight 1985	2B	0.42	1.35	1.47	2 #4hoops	34.97	414	0.4	0.4
Ehsani & Wight 1985	3B	0.39	1.07	1.6	3 #4hoops	40.90	414	0.25	0.25
Ehsani & Wight 1985	4B	0.37	1.41	1.47	3 #4hoops	44.64	414	0.6	0.6

(1) column axial load N
(2) aspect ratio
(3) average yield strength fy of longitudinal reinforcement
(4) approximate values extracted from published diagrams

f_c : compressive strength of concrete
A_c: column section
M_{Rc} : flexural strength of column
M_{Rb} : flexural strength of beam
α_p, α_n: pinching factor for positive and negative loading direction, respectively



(a) Analytical model for the evaluation of the exterior joint initial stiffness



(b) Analytical results and comparison with the experimental

Fig. 3 Evaluation of the initial stiffness of the joint element (first elastic cycle of loading). Comparison with experimental results (specimen J0 by Karayannis *et al.* 2003). For an elastic response the unloading curve of the model returns to zero. Experimental curve has a small residual deformation

comparisons with the experimental results indicated that the initial elastic stiffness of the joint element model can be evaluated following the next procedure:

(a) An elastic analysis of the RC joint subassemblage is performed with the portions of the beam and columns inside the joint to be simulated as inelastic elements (no rigid zones) with the real mechanic characteristics of the materials for each of the beam and columns in the joint.

(b) A new elastic analysis of the RC beam-column joint subassemblage is performed with the portions of the beam and columns inside the joint area this time to be simulated as rigid zones. In this analysis the rotational joint element is included in the analysis model.

(c) The most suitable initial stiffness for the joint element model in (b) is the one that yields results that fit best the results of the model (a).

An indicative example from the implementation of this procedure in order to estimate the initial stiffness of the exterior joint element for the simulation of specimen J0 by Karayannis *et al.* (2003) is presented in Fig. 3(b). In fact the elastic response of specimen J0 was first approached by employing in the analysis model inelastic elements with the real characteristics of the materials for the simulation of the portion of the beam and the columns in the core area of the joint. The results of this analysis are presented in Fig. 3(b) as “approach J0_3”. Using the proposed approach for modeling the RC joint, the most suitable initial stiffness for the joint element model by Favvata *et al.* (2008) is estimated to be equal to 11.430 KNm/rad (“analysis”). For refined analysis in both analytical models for the simulation of the beam and the columns were used 20 inelastic elements (length of each element: 5 cm) and 15 inelastic elements (length of each element: 5 cm), respectively (Fig. 3(a)). Comparison of the above analytical results with the corresponding experimental data results demonstrates the effectiveness of the proposed procedure.

The described procedure has been applied in the present study for the evaluation of the initial stiffness of the exterior beam-column joints.

4. Ultimate strength of the joint

For the purposes of this study the analytical approach presented by Favvata *et al.* (2008) is adopted for the definition of the ultimate strength of the exterior beam - column RC joints. This evaluation procedure involves the following three basic failure mechanisms: (i) the shear strength of the joint, (ii) the development of the maximum bond stress along the horizontal part (ℓ_{sp}) of the beam longitudinal reinforcement inside the core area of the joint and (iii) the flexural yield strength of the adjacent beam. The type of failure mode and therefore the ultimate strength of the exterior joint are estimated by the most critical failure mechanism. The joint ultimate strength is defined in terms of an equivalent flexural strength (M_{Rj}) for direct implementation to the rotational spring element model that is commonly used for the simulation of the joint’s local response.

Shear strength of the joint: For the evaluation of the shear strength of the joint the iterative procedure that has been suggested by Favvata *et al.* (2008) can be used. This procedure involves the following considerations: (a) the joint reaches its maximum strength when the maximum compressive stress in the direction of the diagonal strut reaches the ultimate compression strength of the concrete, (b) the average principal stress of the concrete is employed in the direction of the diagonal strut and is calculated from compatibility conditions (Hsu 1993), and (c) the ultimate compression strength of concrete including the softening effect is established based on the softened stress-strain relationship proposed by Belarbi and Hsu (1995). Nevertheless, as it has already reported the above mentioned procedure has been reconsidered and supplemented (Karayannis *et al.* 2011) based on the theoretical considerations of the well established beam-column joints model by Tsonos (2007). According to Tsonos model the evaluation of the ultimate shear strength of the joints is based on the strut-and-tie mechanism. Considering that the ultimate concrete strength of the joint under compression/tension controls the ultimate strength of the connection the ultimate shear strength of the joint is defined by the solution of a fifth-order polynomial equation taking into account the biaxial concrete strength. Once the shear strength of the joint has been estimated the corresponding equivalent moment (M'_{Rj}) that is developed in the adjacent beam can be calculated.

Bond stress inside the core area of the joint: The analytical procedure that incorporates the influence of the yield penetration on the ultimate capacity of the RC joints due to unfavorable bond conditions is herein presented. In fact, this procedure indicates that the primary causes of damage are two; either the inadequacy of the strength of the diagonal strut or the flexural yielding of the beam. Nevertheless, the final type of failure mode is evaluated by taking into account the level of the developed bond stress (τ) in the horizontal straight part of the anchored beam's bar inside the area of the joint. If this is the critical case diagonal tension cracking is expected to occur due to bond slip of the bar inside the core of the joint. The corresponding bond stress to the maximum strength of the diagonal strut might also be estimated for comparison purposes. Indicative results about the influence of the yield penetration on the capacity of the horizontal part of anchorage in the joint area are presented in Fig. 4 in terms of bond stress vs. slip for the specimen J0 by Karayannis *et al.* (2003). The bond-slip capacity values provided by CEB-FIP MC90 are also shown in Fig. 4 for comparison reasons. In brief the following states are taken into account (more details can be found in Favvata *et al.* 2008):

(1) No flexural yielding of the beam: In this case, the critical level of bond stress is that developed in the first horizontal part of anchorage in the elastic stage (ℓ_{sp}), and the effectiveness of the horizontal part of the anchorage is checked at the maximum bond stress (τ_{max}). In this case the developed force of the longitudinal bar of the adjacent beam is

$$T_{\tau 1} = \ell_{sp} \cdot \pi \cdot \varnothing \cdot \tau_{max} \tag{1}$$

and the corresponding equivalent moment ($M_{R\tau}$) can be calculated.

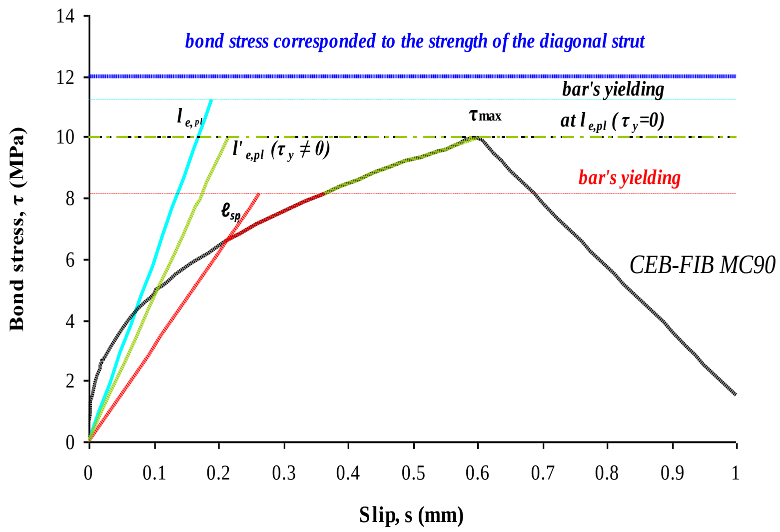
(2) Flexural yielding of the beam: Due to a possible extension of the bar yielding area and its penetration into the joint area, the length of the anchorage that remains in the elastic stage is reduced. Assuming that the extension of the bar yield length is in the range of 5ϕ (ϕ: bar's diameter) and that the post-yield bond stress is zero ($\tau_y = 0$), the force of the longitudinal bar of the adjacent beam ($T'_{\tau 1}$) corresponding to the maximum bond stress ($\tau_{\max}$) is calculated as:

$$T'_{\tau 1} = (\ell_{sp} - 5\varnothing) \cdot \pi \cdot \tau_{max} \tag{2}$$

the corresponding critical equivalent moment developed in the adjacent beam is $M'_{R\tau}$.

A different approach that includes the post-yield bond stress ($\tau_y \neq 0$) is also considered. In this approach, the definition of the extension of the bar's yield length (ℓ_y) in the core area of the joint is considered first. The solution procedure involves the estimation of the critical length ($\ell_{cr} = \ell_y$) and the calculation of the post-yield bond stress (τ_y) developed along the horizontal part of the bar, once the ultimate slip (s_{ult}) at the maximum bond stress is reached. In this way, the reduced horizontal part ($\ell_{sp} - \ell_{cr}$) of the anchorage that remains in the elastic stage, and the stress of the bar (f_s) are defined. The developing force of the longitudinal bar of the adjacent beam is:

$T_s = A_s \cdot f_s$, and the corresponding critical equivalent moment developed in the adjacent beam is $M_{s\tau}$.



Specimen J0 (Karayannis *et al.* 2003)

$\ell_{e,pl} = \ell_{sp} - 5\phi$ (mm)	130
$M'_{R\tau}$ (KNm)	21.14
$\min\{M_{Rb}, M'_{R\tau}\}$ (KNm)	21.14
$\ell'_{e,pl} = \ell_{sp} - \ell_{cr}$ (mm)	146.60
τ_E (Mpa)	9.96
s_E (mm)	0.214
s_{ult} (mm)	0.6
$\ell_{cr} = \ell_y$ (mm)	33.4
τ_y (Mpa)	2.585 ($0.45\sqrt{f_c}$)
$M_{s\tau}$ (KNm)	25.24

Diagonal tension cracking is expected to be developed in the joint after yielding of the adjacent beam. This is attributed to the fact that the yield penetration in the area of the joint results to the development of a high level bond stress before the diagonal strut reaches the maximum strength.

Fig. 4 Influence of the yield penetration on the capacity the capacity (bond stress –slip) of the horizontal straight part of anchorage in the core area of the joints

Considering the results presented in Fig. 4 it can be observed that after beam's yielding the extension of the bar's yield area in the joint's core, substantially increases the possibility of diagonal tension cracking in the joint area. Further the bond stress that corresponds to the maximum strength of the diagonal strut is also presented. In fact diagonal tension cracking is expected to be developed in the joint after yielding of the adjacent beam. This is attributed to the fact that the yield penetration in the area of the joint results to the development of a high level bond stress before the diagonal strut reaches the maximum strength.

Nevertheless the application of the above mentioned procedure in the case of specimen 4B by Ehsani and Wight 1985 indicated that the level of the bond stress that corresponds to the yielding of the adjacent beam and the level that corresponds to the maximum strength of the diagonal strut are almost the same. Thus, diagonal strut failure is expected to occur in the joint after the yielding of the adjacent beam.

It is noted that the values of the post-yield bond stress, τ_y, that are evaluated based on the proposed approach are in a reasonable agreement with the corresponding values proposed in the literature where it is reported based on experimental data that these values are in the range of $0.4\sqrt{f_c}$ - $0.05\sqrt{f_c}$ (Lowes and Altoontash 2003). See also Eligehausen *et al.* (1983) and Shima *et al.* (1987).

5. Pinching factors

An experimental data bank extracted from literature (Table 1) is reported for the real range of the pinching level as observed in a series of tested specimens. Twenty two experimental results were taken into account in order to acquire experimental experience of the range of the real levels of pinching effect. The factors α_p and α_n are defined herein for the characterization of the level of the pinching effect in positive and in negative loading directions, respectively as follows (Fig. 2):

$$\alpha_p \text{ or } \alpha_n = \frac{\text{loading capacity at the predefined deformation } \theta_{pp} \text{ or } \theta_{pn} \text{ with pinching effect}}{\text{loading capacity at the predefined deformation } \theta_{pp} \text{ or } \theta_{pn} \text{ without pinching effect}}$$

In this way the level of the pinching effect on the hysteresis response of the real joints can be estimated. The selected specimens have different characteristics such as: axial load ratio, flexural strength ratio (columns to beam), joint aspect ratio and reinforcement inside the joint core area. The determination of the pinching factors α_p and α_n using the experimental data and the properties of the joints are presented in Table 1.

Based on these results it can be observed that in all the examined cases the pinching factors are approximately between the values of 0.2 to 0.6 and thus two characteristic limit levels of the pinching effect might be considered:

(a) High level of pinching effect when the pinching factors (α_p and α_n) are equal to 0.2 which means 80% less loading capacity at the predefined rotational deformation (θ_{pp}, θ_{pn}).

(b) Low level of pinching effect when the pinching factors (α_p and α_n) are equal to 0.6 which means 40% less loading capacity at the predefined rotational deformation (θ_{pp}, θ_{pn}).

The observed values of the pinching factors for positive (α_p) and negative (α_n) loading direction are almost the same in the examined cases. Nevertheless in the exterior joints where the top and the bottom beam reinforcement anchorages are not the same different values of pinching factors between α_p and α_n are extracted. This type of pinching effect is observed in the specimens "*Unit1*" and "*Unit6*" in Pantelides *et al.* (2002).

The experimental data examined in this study also indicate that the so-called predefined rotational deformations (θ_{pp}, θ_{pn}) at which pinching is occurs are usually between zero deformation and the deformation that corresponds to "yield" points. An approximate value might the deformation corresponding to one-half of the one at "yield" point.

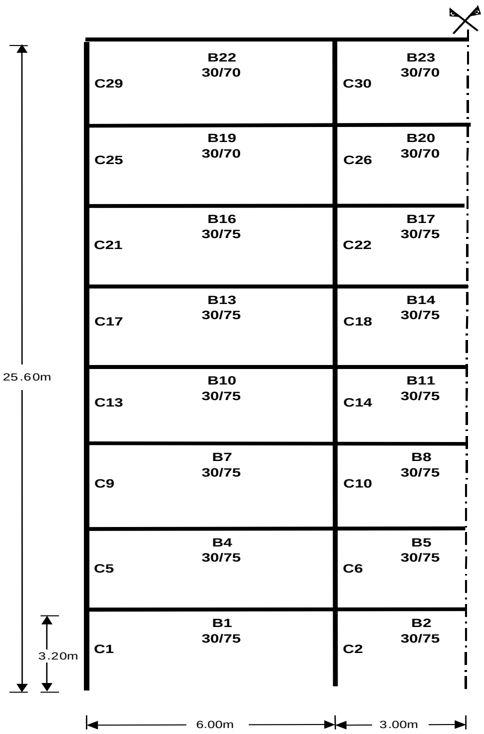
6. Influence of pinching on the seismic response of multistory RC structures

Nonlinear dynamic analyses have been performed in order to investigate the influence of the local response of the exterior beam-column joints with strength degradation, stiffness degradation and pinching effect on the seismic behavior of multistory RC frame structures. Further, the effect of the exterior joints with reduced capacity and hysteresis pinching characteristics on the seismic response of an infilled 8-storey frame structure without infills at the base floor (pilotis type building) is also studied.

The geometry of the considered 8-storey frame and reinforcement of its columns are shown in Fig. 5. For the simulation of the inelastic responses of beams and columns a special quartic element (Izzuddin *et al.* 1994, Karayannis *et al.* 1994) is used while the equivalent diagonal strut model is adopted for the simulation of the behavior of the infill panels. More details about the assumptions of the structural modeling can be found in a previous work by Karayannis *et al.* (2011).

For the purpose of this study all the exterior joints of the examined frames have reduced capacity whereas three different types of hysteretic response are considered: (a) joints with strength and stiffness degradation characteristics without pinching effect, (b) joints with strength degradation, stiffness degradation and low pinching effect and (c) joints with strength degradation, stiffness degradation and high pinching effect. Using the experimental data that were presented in section 5 two different levels of pinching factors are taken into account:

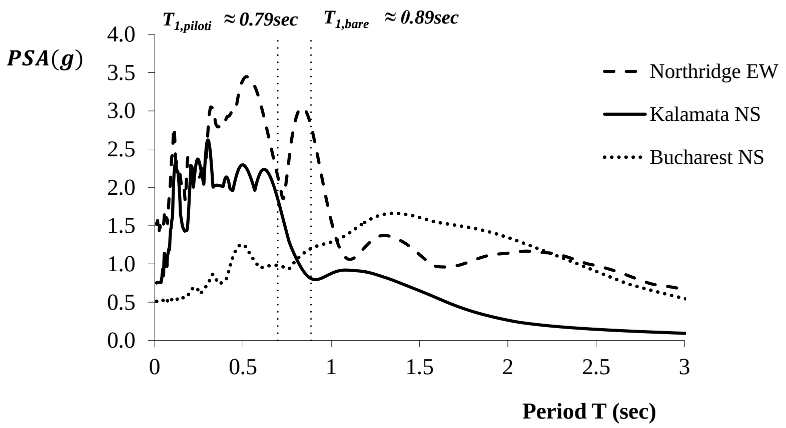
(a) pinching factors (α_p, α_n) equal to 0.6 which means 40% less capacity at the predefined rotational deformations θ_{pp}, θ_{pn} characterized herein as low pinching effect, and
(b) pinching factors (α_p, α_n) equal to 0.2 which means 80% less capacity at the predefined rotational deformations θ_{pp}, θ_{pn} characterized as high pinching effect. Moreover, results for the case that the exterior joints are considered as well designed joints without reduced capacity (rigid exterior joints) are also presented for comparison purposes.



External columns	reinforcement	*Internal columns*	reinforcement
C29 35/30[cm]	6Ø14 [mm]	**C30** 35/35	8Ø14
C25 35/30	4Ø20+2Ø14	**C26** 35/35	6Ø20 up 4Ø20+4Ø16 dn
C21 35/30	4Ø20+2Ø16	**C22** 40/40	10Ø20 up 4Ø20+6Ø14 dn
C17 35/30	4Ø20+4Ø18 up 4Ø20+4Ø16 dn	**C18** 40/40	12Ø20 up 8Ø20+4Ø18 dn
C13 40/35	8Ø20+2Ø14	**C14** 45/45	8Ø20+2Ø16
C9 50/45	4Ø20+4Ø16	**C10** 45/45	8Ø20+4Ø18
C5 50/50	8Ø20	**C6** 50/50	8Ø20+2Ø16 up 8Ø20+2Ø18 dn
C1 50/50	8Ø20 up 8Ø20+2Ø14 dn	**C2** 50/50	8Ø20 up 12Ø20 dn

Fig. 5 Geometry and columns reinforcement of the 8-storey RC frame structure

Response Spectra of the selected earthquake records



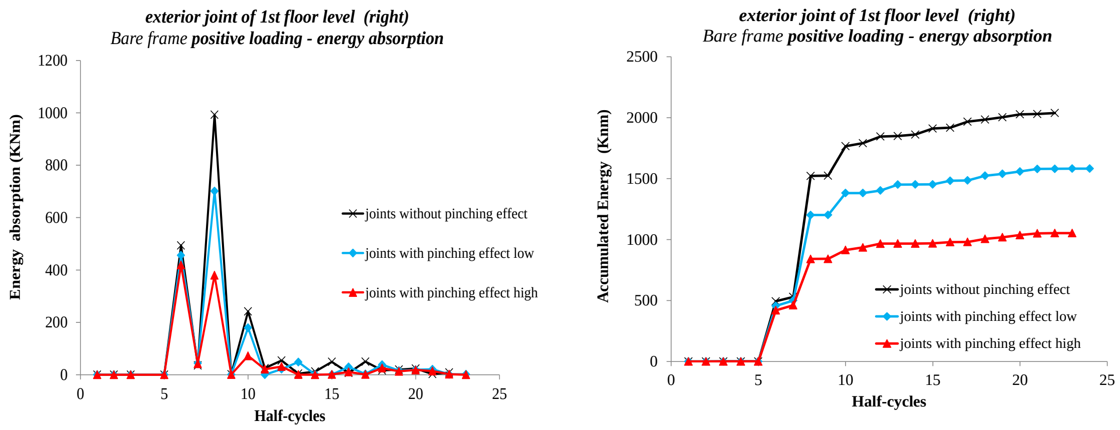
Earthquake	*Magnitude*	*Station*	*Component*	*PGA (g)*
Northridge, USA (17/1/1994)	5.13	Sylmar County Hospital	EW	0.604
Kalamata, Greece (13/9/1986)	5.8	Kalamata	NS	0.297
Bucharest, Romania (4/3/1977)	7.3	Bucharest	NS	0.202

Fig. 6 Seismic excitations and corresponding response spectra

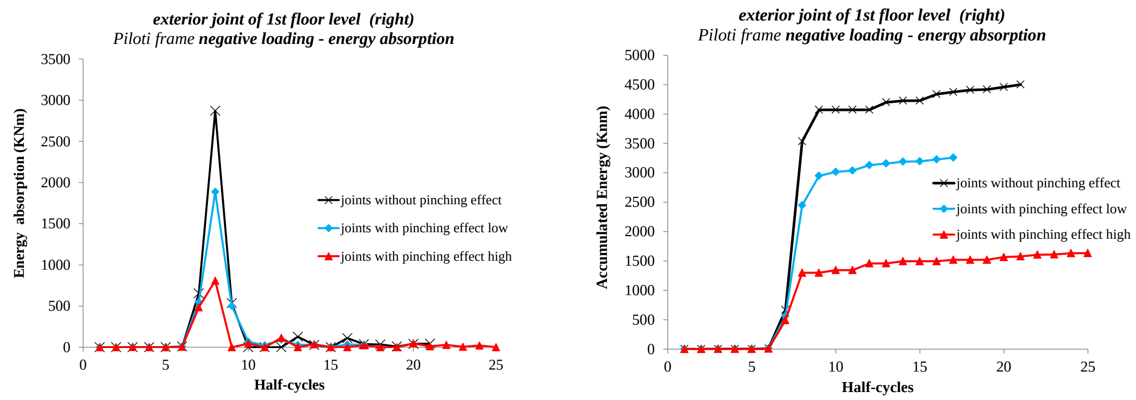
Three different seismic excitations are used in the dynamic analyses (see Fig. 6). Considering that the examined frames are subjected to strong seismic actions the maximum acceleration of these excitations has been scaled to $\alpha_{\max}$ = 0.45 g which is 1.5 times the design acceleration of the examined structures.

In Fig. 7 comparative results of the absorbed energy of the exterior joints of the 1st floor level for the case without pinching and for the cases that includes pinching are presented. These results concern the response of the exterior joints of bare frame and pilotis type frame structures during the seismic excitation of Kalamata. The accumulated energy per half-cycle of the same examined joints is also presented in Fig. 7. It can be observed:

(a) As it is expected the ability for energy absorption is clearly reduced due to pinching effect. The maximum accumulated energy absorption per half-cycle was reduced by 48.33% for the exterior joint of the 1st storey (right) of the bare frame (in positive direction) and 63.50% in the case of pilotis frame 1st storey joint (right - negative direction) due to pinching. When the pinching effect is considered low the corresponding values of energy absorption of the joints lay between the other two examined cases (without pinching effect and with high pinching effect). For instance, the exterior joint of the 1st floor level of the bare frame lost about 22.37% of its maximum accumulated capacity for energy absorption (positive loading) due to low pinching effect.



(a) Bare RC frame structure



(b) Pilotis type RC frame structure

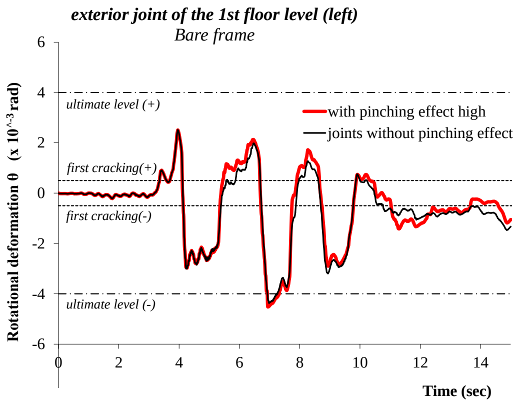
Fig. 7 Influence of pinching on the capacity of the exterior joints for energy absorption per half-cycle of loading. The results are for the case of bare and pilotis type RC structures during the seismic excitation of Kalamata, Greece

(b) The pinching in the direction of loading opposite to the one that causes strength degradation in the joints influences the overall capacity of the joints for energy absorption.

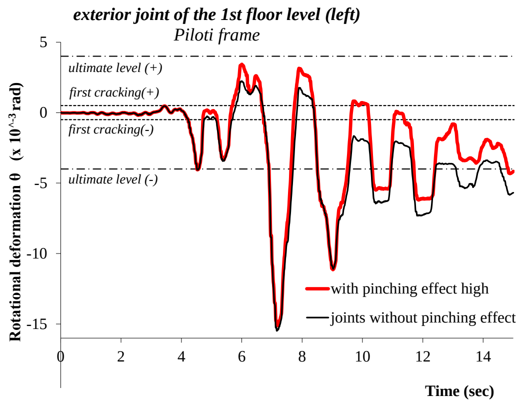
(c) The number of hysteretic cycles is changed due to the pinching effect.

Similar conclusions are also drawn from the results for the case of Northridge seismic excitation. In this case the maximum accumulated energy per half-cycle in positive loading direction of the exterior joint of the 1st floor level of the bare frame was 36.73% less than the corresponding capacity of it without the pinching effect, while for the same joint of pilotis type frame the loss was 56.96%. In the negative loading direction the influence of pinching on the developed maximum accumulated energies per half-cycle was significantly lower (e.g. the loss was 8.76% for the exterior joint of the 1st floor of the bare frame and almost zero for the corresponding joint of pilotis). Further the reduced ability for energy absorption of these joints due to pinching effect occurred at early stage of the seismic loading (5th half-cycle in case of bare frame and 8th half-cycle in case of pilotis frame).

Time histories of the rotational deformation responses of the exterior joints for the first floor levels of bare frame and pilotis type frame are presented in Fig. 8 in an effort to study the influence of pinching on the deformation demands of the joints. In this figure the time history requirements for the seismic excitations of Northridge (USA) and Kalamata (Greece) are presented. It can be observed that for the examined joints of the bare frame structure damages have occurred inside the joints core area. However the ultimate deformation level has not been exceeded throughout the Kalamata excitation while in case of Northridge limited strength degradation characterize the seismic response of the 1st floor level exterior joint. The influence of the pinching

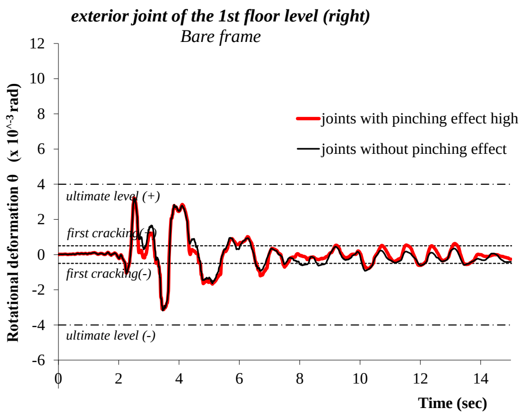


(a)

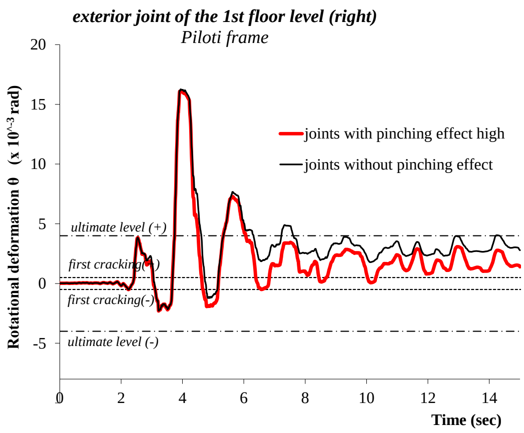


(b)

(i) Seismic excitation Northridge, USA



(c)



(d)

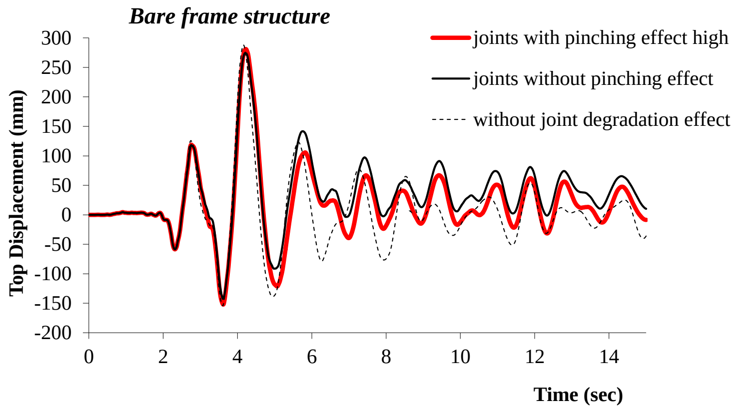
(ii) Seismic excitation Kalamata, Greece

Fig. 8 Influence of pinching on the time histories of the rotational deformation responses of the exterior joints of the first floor level of bare and pilotis type RC structures during the seismic excitations of Northridge,USA & Kalamata, Greece during the hysteretic responses of the exterior joints (1st floor of bare frame) on their rotational demands for deformation is proved to be negligible.

On the contrary the exterior joints at the first floor of pilotis type structure developed high demands for deformation that exceed the ultimate capacity level during both seismic excitations (Figs. 8(b) and 8(d)). In fact the seismic behavior of these joints has shown characteristics of strength degradation declaring that severe damages or even failures have occurred. High deformation demands were also observed at the exterior joints of the 7th level of the bare frame. Focusing on the pinching parameter it can be deduced that the rotational deformation responses of the exterior joints are influenced by the pinching effect if the joints exhibit strength degradation as well.

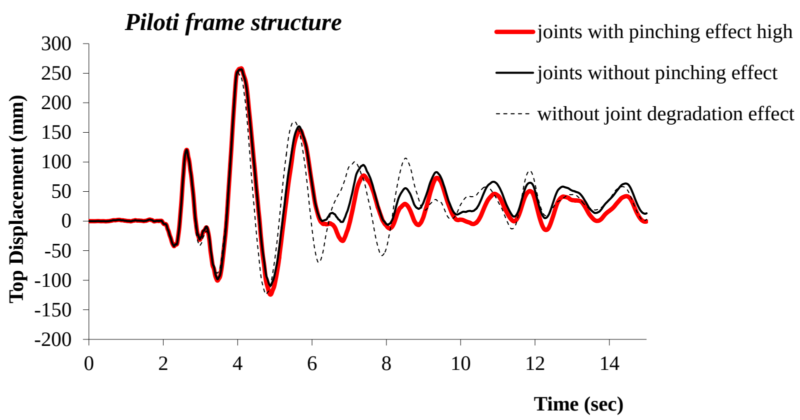
In Fig. 9 results of the influence of pinching on the time histories of the top displacement of the examined structural systems are presented for the seismic excitation of Kalamata. It is observed that in the negative direction of loading the developing top displacements of both examined structures are somewhat greater in the case that includes pinching in the responses of the exterior joints compared to the ones without pinching. Nevertheless in the positive direction an insignificant reduction on the corresponding top displacements of the structures is observed due to pinching effect. Similarly, a small increase of the displacements of the bare frame in the positive direction is observed due to the pinching effect in the case of Northridge excitation. On the opposite side (negative direction) the developing top displacements of the bare frame are somewhat greater in the case that includes pinching in the hysteretic responses of the joints compared to the ones without pinching. Nevertheless the influence of the pinching on the top displacements of pilotis type frame during the Northridge excitation is negligible.

Based on the results of this study it can be noted that the effect of pinching on the local and global responses of the examined cases is limited in the cases where high level strength degradation in the area of exterior joint has occurred. This conclusion can also be deduced from the results of the influence of the pinching on the maximum interstory drifts (for negative displacements) of the bare frame. In this case the requirements for interstory drift of the third floor level are increased due to the pinching when compared to the ones without considering the pinching in the joint's responses. Studying the hysteretic response of the exterior joint of the 3rd



(a)

Continued



(b)

Fig. 9 Influence of pinching on the time histories of the top displacement of the bare and pilotis type RC structures during the seismic excitation of Kalamata, Greece

floor level of the same structure it is observed that no strength degradation occurred in the joint in both cases; with and without pinching effect. It is also stressed that in the positive displacements the effect of pinching was negligible.

Finally in this study the effect of the pinching on the maximum curvature ductility demands (μ_φ) of the columns at the base of the bare and pilotis type building is also examined and presented in Table 2 for the seismic excitations of Kalamata, Northridge and Bucharest.

The results presented are for the cases that all the exterior joints of the examined frames have reduced capacity whereas three different rules of hysteretic response were considered: (a) joints with strength and stiffness degradation without pinching effect, (b) joints with strength degradation, stiffness degradation and with low pinching effect and (c) joints with strength degradation, stiffness degradation and with high pinching effect. Further in order to investigate the effect of pinching in this study two different levels were taken into account: (a) low level of pinching effect ($\alpha_p = \alpha_n = 0.6$; 40% less capacity at the predefined rotational deformations $\theta_{pp}= 1/2\theta_{yp}$ and $\theta_{pn} = 1/2\theta_{yn}$) and (b) high level of pinching effect ($\alpha_p=\alpha_n=0.2$; 80% less capacity at the predefined rotational deformations $\theta_{pp}= 1/2\theta_{yp}$ and $\theta_{pn}=1/2\theta_{yn}$, see Figs. 1 and 2). Finally results for the case that the exterior joints are considered as well designed joints without reduced capacity (rigid exterior joints) are also presented for comparison reasons.

It can be deduced that almost in all the examined cases the maximum demands for curvature ductility of the columns at the base are decreased when the exterior joints of the structures are considered with reduced capacity compared with the corresponding demands of the columns of the structures with all the joints to be rigid. Of course this is not a benefit since it implies uncontrollable damages and failures in the joints core area at the 1st floor level (see also Karayannis *et al.* 2011). Severe damages and failures at the exterior joints are making the pinching effect to be negligible on the developing ductility requirements of the columns at the base compared with the corresponding demands of the columns without joints pinching effect. Thus it can be deduced that significant changes on the curvature ductility demands of the columns at the base of the structures (with reduced capacity exterior joints) are not expected to occur due to pinching. Nevertheless pinching characteristics might increase the demands for ductility of the

Table 2 Influence of the pinching on the maximum curvature ductility demands (μ_φ) of the columns at the base of the structures for the several seismic excitations

Maximum curvature ductility demands (μ_φ) of the columns of Bare RC 8-storey frame structure Kalamata, Greece				
Column	exterior joints hysteretic response			
	without joint degradation	without pinching	with pinching effect low	with pinching effect high
C1	3.81	2.34	2.51	2.32
C2	2.10	1.74	1.82	1.78
C3	1.18	1.05	1.02	1.03
C4	elastic	elastic	elastic	elastic
Maximum curvature ductility demands (μ_φ) of the columns of Pilotis type RC 8-storey frame structure Kalamata, Greece				
Column	exterior joints hysteretic response			
	without joint degradation	without pinching	with pinching effect low	with pinching effect high
C1	5.93	3.54	3.62	3.61
C2	4.5	4.28	4.17	4.02
C3	6.22	6.44	6.50	6.60
C4	13.79	7.42	7.26	6.99
Maximum curvature ductility demands (μ_φ) of the external column C1 of RC 8-storey frame structures Northridge, USA				
Structure	exterior joints hysteretic response			
	without joint degradation	without pinching	with pinching effect low	with pinching effect high
Bare frame	1.002	1.42	1.43	1.56
Piloti frame	10.62	4.92	5.19	4.89

Maximum curvature ductility demands (μ_φ) of the external column C4 of RC 8-storey frame structures Bucharest, Romania			
Structure	exterior joints hysteretic response		
	without joint degradation	without pinching	with pinching effect high
Bare frame (demands until 3.7 sec)	36.81	22.04	22.11
Piloti frame (demands until 3.5 sec)	31.71	18.64	18.68

columns in case when strength degradation is not exhibited by the joints (early stage of seismic loading). This type of effect is deduced for the external column of the bare frame in case of Northridge excitation and it is depicted in Fig. 10 (see also Table 2). However in this case the demands for curvature ductility of the column are not critically increased due to pinching.

7. Conclusions

Nonlinear dynamic analyses were carried out in order to investigate the influence of the pinching hysteretic response of the exterior RC beam-column joints on the seismic behavior of

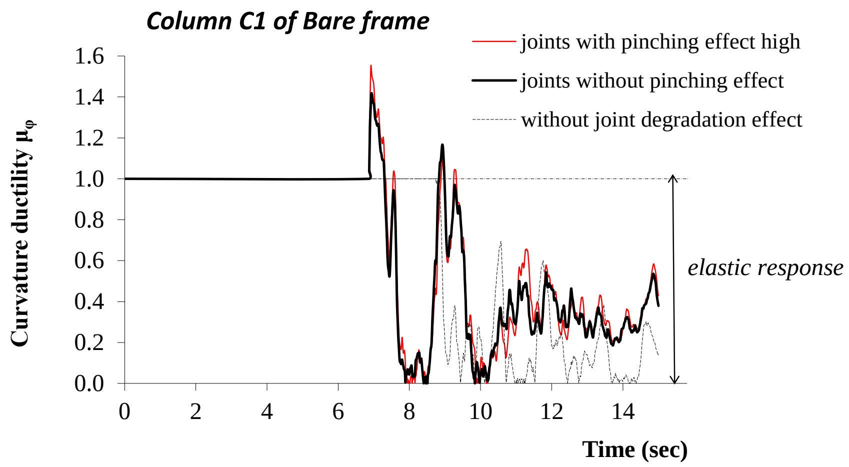


Fig. 10 Influence of pinching on the developing demands for curvature ductility of the external column C1 of the bare frame RC structure during the seismic excitation of Northridge, USA

multistory RC frame structures. For the purposes of this study three different types of hysteretic response were considered: (a) joints with strength and stiffness degradation characteristics but without pinching effect, (b) joints with strength degradation, stiffness degradation and low pinching effect and (c) joints with strength degradation, stiffness degradation and high pinching effect. The values of the pinching factors that determine the levels of the pinching effect were estimated based on a data bank of reported tests from literature. Results showing the influence of pinching on the local and global mechanisms of an 8-storey bare frame and an 8-storey pilotis type frame structure were presented in terms of hysteretic responses of the joints, joints capacities for energy absorption, maximum accumulated energies, rotational requirements of the joints responses, curvature ductility requirements of columns, requirements for top displacement of the structures and interstory drifts. Based on the results of this study the following concluding remarks may be drawn:

- As it was expected the ability for energy absorption was reduced due to pinching effect.
- The number of hysteretic cycles was changed due to pinching effect.
- In general the effect of pinching on the local and global responses of the examined frames was not significant in the cases when significant strength degradation of exterior joint occurred.
- The pinching in the direction of loading opposite to the one that causes strength degradation in the joints was influencing the overall capacity of the joints for energy absorption.
- The demands for rotational deformation of the exterior joints were influenced by the pinching effect only when the joints exhibited strength degradation as well, otherwise the influence of the pinching effect was negligible.
- Similarly, the global requirements (time histories of the top displacement and interstory drifts) of the examined structural systems were affected due to the pinching only in the cases when strength degradation inside the core area of exterior joint did not take place.
- Finally, in this study the effect of the pinching on the maximum curvature ductility demands (μ_φ) of the columns at the base of the bare and pilotis type building was investigated. The results

indicated that almost in all the examined cases the maximum demands for curvature ductility of the columns at the base are decreased when the exterior joints of the structures are considered with reduced capacity compared with the corresponding demands of the columns of the structures with all the joints to be rigid. Of course this is not a benefit since it implies uncontrollable damages and failures in the joints core area at the 1st floor level (see also Karayannis *et al.* 2011). Severe damages and failures at the exterior joints are making the pinching effect on the ductility requirements of the columns at the base to be negligible compared with the corresponding demands of the columns without joints pinching effect. Thus it was deduced that significant changes on the developed curvature ductility demands of the columns at the base of the examined structures are not expected to occur due to pinching. Nevertheless the results also shown that the pinching response may increase the demands for ductility of the columns in the case when strength degradation is not exhibited by the joints.

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