

Article

Not peer-reviewed version

Probabilistic IM–Based Assessment of Critical Engineering Demand Parameters to Control the Seismic Structural Pounding Consequences in Multistory RC Buildings

[Maria J. Favvata](#) * and [Effrosyni G. Tsiaga](#)

Posted Date: 17 December 2025

doi: 10.20944/preprints202512.1455.v1

Keywords: structural pounding; engineering demand parameter; damage mitigation; separation distance; probability of pounding; performance levels; fragility curve; PSDM; nonlinear dynamic analyses; EC8



Preprints.org is a free multidisciplinary platform providing preprint service that is dedicated to making early versions of research outputs permanently available and citable. Preprints posted at Preprints.org appear in Web of Science, Crossref, Google Scholar, Scilit, Europe PMC.

Copyright: This open access article is published under a [Creative Commons CC BY 4.0 license](#), which permit the free download, distribution, and reuse, provided that the author and preprint are cited in any reuse.

Disclaimer/Publisher's Note: The statements, opinions, and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions, or products referred to in the content.

Article

Probabilistic IM—Based Assessment of Critical Engineering Demand Parameters to Control the Seismic Structural Pounding Consequences in Multistory RC Buildings

Maria J. Favvata ^{1,*} and Effrosyni G. Tsiaga ²

¹ University of Patras, Civil Engineering Department, Rio 26504, Greece

² Civil Eng. M.Sc., University of Patras, Civil Engineering Department, Rio 26504, Greece

* Correspondence: mfavvata@upatras.gr

Abstract

This research aims to assess and quantify the significance of incorporating the seismic performance of global and local engineering demand parameters (EDPs) within the probabilistic frameworks, when structural pounding of adjacent buildings occurs. For this purpose, the seismic performance of 6-story and 12-story reinforced concrete (RC) RC frames subjected to floor-floor pounding is assessed. The pounding is caused by an adjacent shorter and stiffer structure with the top contact point at the middle of the tall building's total height. Displacement-based and ductility-based EDPs are evaluated at different performance levels (PLs) and at different separation distances (d_g). The seismic performance of the RC frames without considering pounding is also evaluated. Incremental dynamic analyses (IDAs) are performed, and probabilistic seismic demand models (PSDMs) are developed to establish fragility curves of the examined RC frames. The probability of earthquake-induced pounding between adjacent structures is properly involved with the median value of $S_{a,T1}$ that corresponds to an acceptable capacity level (acceptable PL) of an EDP. The results of this study indicate that excluding structural pounding consequences from the probabilistic frameworks related to the seismic risk of colliding buildings leads to unsafe seismic assessment or design provisions.

Keywords: structural pounding; engineering demand parameter; damage mitigation; separation distance; probability of pounding; performance levels; fragility curve; PSDM; nonlinear dynamic analyses; EC8

1. Introduction

An important issue in earthquake engineering that several researchers have focused on is the interaction of buildings in proximity. In these studies, the structural seismic response during pounding impacts was primarily estimated based on the results of nonlinear time histories analyses (deterministic assessment). Several key parameters have been addressed to verify the structural performance, such as the type of structural pounding, buildings height, buildings arrangement, separation gap distance, building's material (i.e. reinforced concrete or steel), soil-structure interaction, setbacks, masonry infills, height difference of the colliding structures, the beam-column joint damage effect, mitigation strategies, characteristics of the seismic excitations, among others. Additionally, it has been demonstrated that the seismic performance of buildings that experience collisions depends on both the seismic hazard of the location and the performance criteria established for seismic demands [1].

Advanced concepts in seismic structural engineering incorporate performance-based approaches for the design and the assessment of infrastructure. A key issue in these methods is the

definition of an accurate analytical fragility curve that identifies the vulnerability of structures to seismic damage. In recent years, literature also provides results based on fragility analyses that have been performed to probabilistic evaluation of the structural pounding phenomenon [2–14]. Tubaldi et al. [2] evaluated the seismic pounding risk by developing fragility curves for single-degree-of-freedom (SDOF) and multi-degree-of-freedom (MDOF) systems. The fragility assessment of multistory reinforced concrete or/and steel structures subjected to pounding has also been studied [3–14]. Nazri et al [3] developed fragility curves for different performance levels in terms of maximum interstory drift to study the effect of gaps between RC frame structures and the influence of regularity/irregularity on structural behavior under moderate seismic actions. The structural performance of the adjacent buildings was based on the results of the IDA method as a function of the peak ground acceleration (PGA). The local inelastic demands of the critical structural members into the fragility assessment of buildings subjected to pounding were introduced by Flenga and Favvata [4–9]. Both floor-to-floor and inter-story (floor-to-column) structural pounding were assessed. Additionally, a fragility-based methodology was proposed to evaluate the minimum separation gap distance required at different limit states [7]. Other key issues related to the probabilistic assessment of the structural pounding effect that have been addressed included the investigation of a) the discrepancies between the methodologies used to develop accurate structural seismic fragility curves [4], b) the efficiency and sufficiency of the intensity measures (IMs) [8], and c) the effect of the magnitude M_w and R_{rup} [9]. Mohamed and Romão [10] studied the structural pounding effect on the fragility assessment of non-seismically designed RC structures taking into account both floor-to-floor and floor-to-column pounding conditions. Kazemi et al [11] presented fragility curves to evaluate the impact of floor-to-floor structural pounding at different performance levels on the seismic performance of adjacent RC and steel frames. In 2022, Miari and Jankowski [12] studied the effect of soil type on the fragility assessment of colliding buildings. Sinha and Rao [13] investigated displacement-based fragility for floor-to-floor and floor-to-column collision within an eight-story non-ductile RC frame adjacent to a three-story rigid RC frame. In 2024, Kazemi et al [14] investigated the seismic performance of steel structures in collision with RC structures under floor-to-floor pounding conditions. IDAs were performed and fragility curves in terms of IDR were developed based on M-IDA curves as a function of $S_{a,T1}$.

The review underlines the importance of conducting additional research on the seismic performance-based assessment and design of colliding buildings. There are several important issues that need further investigation to improve understanding and ensure structural safety.

For instance, assessing the fragility results should also extend beyond verifying the severity of the structural pounding phenomenon that causes damage or even collapse. It is essential to identify solutions that ultimately help to define adequate separation gaps and improve risk mitigation strategies. The most popular approach against structural pounding is the evaluation of a sufficiently large separation distance between the adjacent structures. However, although for new buildings it is possible (but not always applicable) to design sufficient gap distances, in the case of existing structures this option cannot be ensured. Mitigation methods to control damage when an insufficient separation distance exists between adjacent buildings are also limited. The consequences of the structural pounding on the damage level of the structure can only be quantified if the performances of critical global and/or local EDPs are properly involved in the probabilistic methods. However, the sole EDP that is usually adopted to evaluate the separation distances is associated with the displacements that are developed in the contact area of the colliding buildings. The same approach is used by the current seismic design building codes, although several studies have proved that these codes' provisions are either inadequate or conservative. Therefore, the code requirement cannot always be applied.

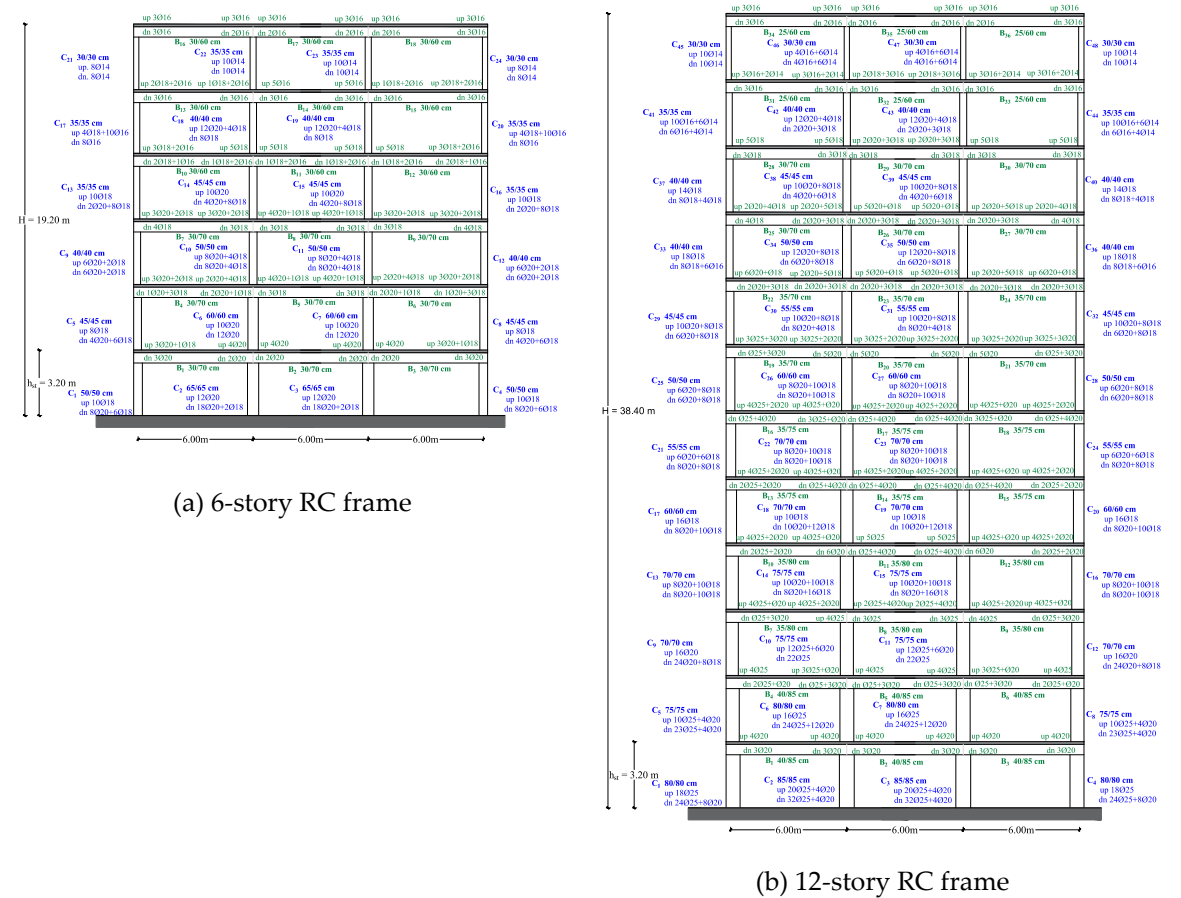
So, this study aims to assess and quantify the significance of incorporating the seismic performance of global and local EDPs within the probabilistic performance-based earthquake engineering frameworks, when structural pounding of buildings occurs.

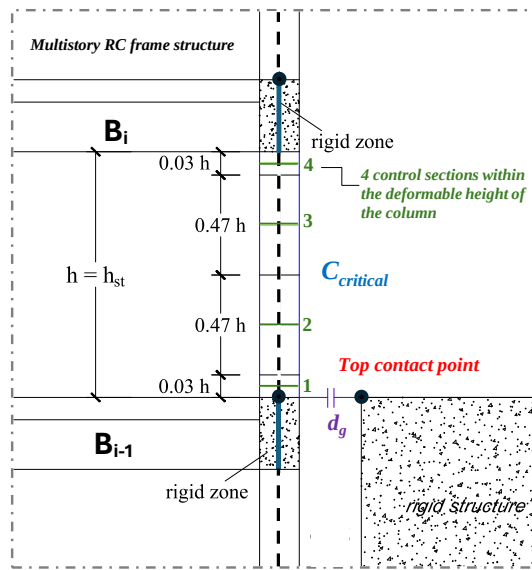
For this purpose, the fragility of two multistory reinforced concrete (RC) frame structures subjected to pounding caused by an adjacent shorter and stiffer structure is studied. Displacement-based and ductility-based EDPs are evaluated at different performance levels and at different separation distances d_g . 1456 nonlinear dynamic analyses are performed based on IDA method and linear and bilinear PSDMs are developed to establish the fragility curves of the examined RC frames.

In this study, the probability of earthquake-induced pounding between adjacent structures is properly involved with the median value of $S_{a,T1}$ that corresponds to an acceptable capacity level (acceptable PL) of an EDP. These results provide insight into the importance of including critical EDPs for predicting and controlling the severe damage caused by seismic pounding between adjacent multistory reinforced concrete buildings.

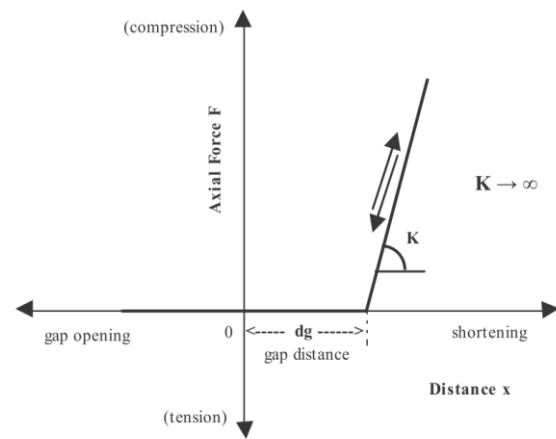
2. Design of Multistory RC Buildings

Two multistory RC frame structures have been designed for the purposes of this work; a) a 6-story, and b) a 12-story structure. All the examined frames were designed according to Eurocodes 2 [15] and 8 [16], meeting the Ductility Capacity Medium (DCM) criteria, while the behavior factor q was equal to 3.75. The mass of the structures was taken equal to $M=(G+0.3Q)/g$ (where G gravity loads and Q live loads) and the design base shear force was equal to $V=(0.3g/q)M$. Reduced values of member moments of inertia (I_{ef}) were considered in the design to account for the cracking; for beams $I_{ef}=0.5I_g$ (where I_g the moment of inertia of the gross section) and for the columns $I_{ef}=0.9I_g$. Critical for the dimensioning of the columns proved to be in most of the cases the code provision regarding the axial load ratio limitation $v_d \leq 0.65$, and in a few cases (columns of the upper stories) the code requirements for minimum dimensions. Structural geometry and reinforcement of the examined multistory RC structures are shown in Figure 1 (a-b).





(c) contact area



(d) contact element response model

Figure 1. Examined multistory RC frame structures (a, b) subjected to floor-to-floor structural pounding at the middle level of the building's total height H , contact area idealization (c), and contact element response model (d).

3. Structural Modelling Assumptions

In this work, the computer program package Drain-2DX is used. The finite element mesh for the RC frame structures utilizes an one-dimensional element for each structural member, while two types of one-dimensional beam-column elements were used. For the simulation of beams the common lumped plasticity model has been employed. For the columns of the structure a special purpose element of "distributed plasticity" type accounting for the spread of inelastic behavior both over the cross-sections and along the deformable region of the member length is adopted (Figure 1c). Collisions are simulated using contact elements that become active when the corresponding nodes come into contact (Figure 1d). This idealization is consistent with the building model used and appears adequate for studying the effects of pounding on the overall structural response for the pounding cases under examination [1,17].

4. Fragility Curve Based on Linear or Bilinear PSDMS

A probabilistic seismic demand model (PSDM) is a mathematical relationship between the median structural demand $\widehat{EDP}$ and the earthquake intensity measure (IM). To develop a PSDM, nonlinear dynamic analyses such as IDA are first performed to generate the EDP-IM pairs, and then a statistical process synthesizes the PSDM. The PSDM is described as:

$$\widehat{EDP}|IM = a IM^b \quad (1)$$

In log-log space the PSDM is expressed as

$$\ln \widehat{EDP}|IM = b \ln IM + \ln a + \varepsilon|IM \quad (2)$$

where, $\varepsilon|IM$ is the random error with mean zero and standard deviation β . The coefficients a and b are calculated through linear regression analysis.

Nevertheless, it has been proved [7] that the local inelastic demands of a structure are described more sufficiently through bilinear regression models (bilinear PSDMs) as:

$$\ln \widehat{EDP}|IM = (a_1 + b_1 \ln IM) (1 - H_1) + (a_2 + b_2 \ln IM) H_1 + \varepsilon|IM \quad (3)$$

where, a_1, b_1, a_2 , and b_2 control the intercepts and the slopes of the first and the second segment of the bilinear model. These coefficients α_1, α_2, b_1 and b_2 are defined by performing a regression analysis with a single error term ($\epsilon | IM$). The H_1 is a dummy variable that when is set equal to zero (0.0) Eq. (3) describes the first segment of the bilinear model, and when $H_1 = 1$ the same equation depicts the second segment. In a bilinear PSDM two different values of dispersion are estimated one for each segment (β_1 for the first segment, and β_2 for the second segment) allowing to relax the homoscedasticity assumption. More details can be found in [7–9].

Thereafter, fragility curves can be estimated based on the closed-form solution of the PSDM as follows:

$$G_{EDP|IM}(C|IM) = P[EDP|IM \geq C|IM] = \Phi\left(\frac{\ln \widehat{EDP}|IM - \ln \hat{C}}{\beta_{EDP|IM}}\right) \quad (4)$$

$\Phi(\cdot)$ denotes the standard normal cumulative function, $\hat{C}$ the median value of the capacity and $\beta_{EDP|IM}$ the logarithm standard deviation calculated as (Eq. 5)

$$\beta_{EDP|IM} = \sqrt{\frac{\sum_{i=1}^n (\ln EDP_i | IM - \ln \widehat{EDP}|IM)^2}{n-2}} \quad (5)$$

n is the number of the nonlinear dynamic analyses.

5. Fragility Curve Based on Linear or Bilinear PSDMS

In this study the floor-to-floor structural pounding is addressed. The potential top contact point is located at the middle floor level of each multistory RC frame structure, meaning at the 3rd floor level in the case of the 6-story RC frame, and at the 6th floor level for the 12-story RC frame. The probabilistic assessment of the multistory RC frames due to the structural pounding effect is performed for three different separation gap distances (d_g) between the adjacent structures: a) $d_g = d_{g,EC8} = q \cdot \delta_{el}$ (Eurocode's requirement), b) $d_g = 0.5d_{g,EC8}$, and c) $d_g = 0.0$ cm (structures are in contact from the beginning). The case of without the pounding effect is also included.

Three EDPs are evaluated, namely: a) maximum displacement δ_{max} at the top of the contact point, b) ratio of maximum interstory drift to the storey height, h_{st} ($IDR_{max} - \%h_{st}$) and c) maximum curvature ductility demands ($\mu_{\phi, max}$) of the most critical external column of the multistory RC frame. Different performance levels for each EDP are considered. In the case of δ_{max} the probability of pounding is estimated taking into account as capacity levels the examined values of separation gap distance d_g . The fragility of the multistory RC structures in terms of $IDR_{max} (\%h_{st})$ is assessed for the two performance levels: (a) Immediate Occupancy (IO) - $IDR_{max, capacity} = 1 (\%h_{st})$, and b) Collapse Prevention (CP) - $IDR_{max, capacity} = 2.5 (\%h_{st})$. Lastly, two capacity levels for the local EDP of $\mu_{\phi, max}$ are considered: (a) Yield capacity state of performance (Y) - $\mu_{\phi, max} = 1.0$, and (b) Ultimate capacity state of performance (U) - $\mu_{\phi, max} = \phi_u / \phi_y$ (ϕ_u = ultimate curvature, and ϕ_y = yield curvature – based on cross-section analysis).

A total number of 1456 nonlinear dynamic step by step analyses are performed based on the IDA method. For this purpose, a set of 14 ground motions extracted from the PEER's database are used, and the spectral acceleration S_{a, T_1} at the fundamental period T_1 of each multistory RC structure ($T_{1, 6-story} = 0.823s$, $T_{1, 12-story} = 1.295s$) is adopted to describe the seismic intensity measure. The characteristics of the selected ground motions (source-to-site distance, magnitude, shear wave velocity, $v_{s, 30}$, etc.) are reported in Flenga and Favvata [7].

6. Results

The probabilistic seismic performance of multistory RC frame structures due to the structural pounding effect is herein evaluated. For this purpose, the PSDMs of $\delta_{max} | S_{a, T_1}$, $IDR_{max} | S_{a, T_1}$, and $\mu_{\phi, max} | S_{a, T_1}$ are initially developed using linear and bilinear regression models, as discussed in Section 4 and shown in Table 1.

Table 1. PSDMs of $\delta_{\max}|S_{a,T1}$, $IDR_{\max}|S_{a,T1}$ and $\mu_{\varphi,\max}|S_{a,T1}$ for the multistory RC frame structures for all the examined cases.

Pounding case	RC frame structure	EDP = aIM ^b		$\beta_{EDP} IM$
without pounding	6-story RC structure	$\delta_{\max} = 0.096 S_{a,T1}^{0.980}$		0.316
	12-story RC structure	$\delta_{\max} = 0.218 S_{a,T1}^{0.913}$		0.291
without pounding	6-story RC structure	$IDR_{\max} = 2.354 S_{a,T1}^{0.978}$		0.286
	12-story RC structure	$IDR_{\max} = 2.939 S_{a,T1}^{0.819}$		0.275
$d_g = 0.0$ cm	6-story RC structure	$IDR_{\max} = 2.835 S_{a,T1}^{0.964}$		0.314
	12-story RC structure	$IDR_{\max} = 4.349 S_{a,T1}^{0.845}$		0.266
$d_g = 0.5 \cdot d_{g,EC8}$	6-story RC structure	$IDR_{\max} = 2.601 S_{a,T1}^{1.005}$		0.322
	12-story RC structure	$IDR_{\max} = 3.633 S_{a,T1}^{0.883}$		0.267
$d_g = d_{g,EC8}$	6-story RC structure	$IDR_{\max} = 2.467 S_{a,T1}^{0.994}$		0.310
	12-story RC structure	$IDR_{\max} = 3.133 S_{a,T1}^{0.838}$		0.284
without pounding	6-story RC structure	$\mu_{\varphi,\max} - C16 = 1.439 S_{a,T1}^{0.836}$	$S_{a,T1} \leq 0.647g$	0.222
		$\mu_{\varphi,\max} - C16 = 1.632 S_{a,T1}^{1.125}$	$S_{a,T1} \geq 0.647g$	0.577
	12-story RC structure	$\mu_{\varphi,\max} - C28 = 1.050 S_{a,T1}^{0.728}$	$S_{a,T1} \leq 0.960g$	0.175
		$\mu_{\varphi,\max} - C28 = 1.075 S_{a,T1}^{1.308}$	$S_{a,T1} \geq 0.960g$	0.325
$d_g = 0.0$ cm	6-story RC structure	$\mu_{\varphi,\max} - C16 = 2.465 S_{a,T1}^{0.914}$	$S_{a,T1} \leq 0.373g$	0.289
		$\mu_{\varphi,\max} - C16 = 7.000 S_{a,T1}^{1.973}$	$S_{a,T1} \geq 0.373g$	0.729
	12-story RC structure	$\mu_{\varphi,\max} - C28 = 2.430 S_{a,T1}^{0.787}$	$S_{a,T1} \leq 0.325g$	0.205
		$\mu_{\varphi,\max} - C28 = 13.929 S_{a,T1}^{2.34}$	$S_{a,T1} \geq 0.325g$	0.684
$d_g = 0.5 \cdot d_{g,EC8}$	6-story RC structure	$\mu_{\varphi,\max} - C16 = 2.173 S_{a,T1}^{0.952}$	$S_{a,T1} \leq 0.443g$	0.168
		$\mu_{\varphi,\max} - C16 = 5.658 S_{a,T1}^{2.128}$	$S_{a,T1} \geq 0.443g$	0.866
	12-story RC structure	$\mu_{\varphi,\max} - C28 = 1.925 S_{a,T1}^{0.882}$	$S_{a,T1} \leq 0.478g$	0.387
		$\mu_{\varphi,\max} - C28 = 28.646 S_{a,T1}^{3.355}$	$S_{a,T1} \geq 0.478g$	1.004
$d_g = d_{g,EC8}$	6-story RC structure	$\mu_{\varphi,\max} - C16 = 1.831 S_{a,T1}^{0.907}$	$S_{a,T1} \leq 0.514g$	0.215
		$\mu_{\varphi,\max} - C16 = 5.083 S_{a,T1}^{2.441}$	$S_{a,T1} \geq 0.514g$	0.970
	12-story RC structure	$\mu_{\varphi,\max} - C28 = 1.426 S_{a,T1}^{0.809}$	$S_{a,T1} \leq 0.660g$	0.290
		$\mu_{\varphi,\max} - C28 = 6.639 S_{a,T1}^{4.513}$	$S_{a,T1} \geq 0.660g$	1.206

Thereafter, the fragility curves of each multistory RC frame structure at different performance levels and different separation distances d_g (including the case of without pounding) are defined (see Figures 2–4).

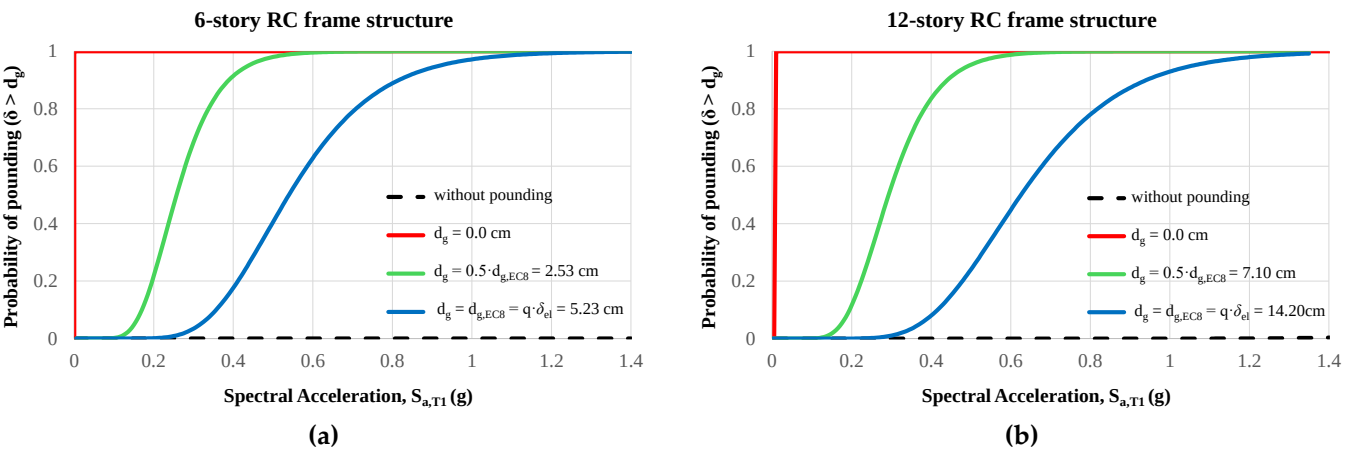


Figure 2. Probability of pounding at different separation distances d_g . (a) 6-story RC frame, and (b) 12-story RC frame.

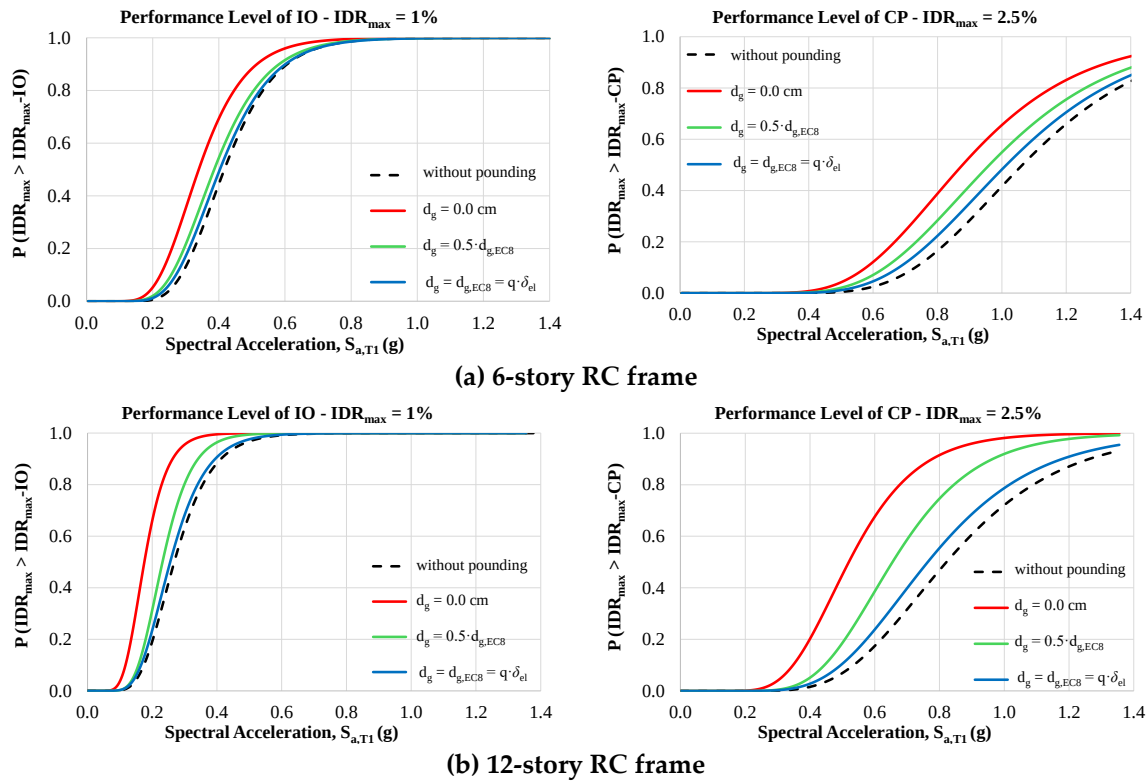


Figure 3. Fragility curves of IDR_{max} at different PLs, and at different gap distances (d_g) for the multistory RC structures subjected to floor-to-floor pounding; (a) 6-story RC frame, and (b) 12-story RC frame. Examined PLs; (1) $IDR_{max} - IO$, and (2) $IDR_{max} - CP$. Results for (i) $d_g = 0.0$ cm, (ii) $d_g = 0.5d_{g,EC8}$, (iii) $d_g = d_{g,EC8}$, and iv) without pounding.

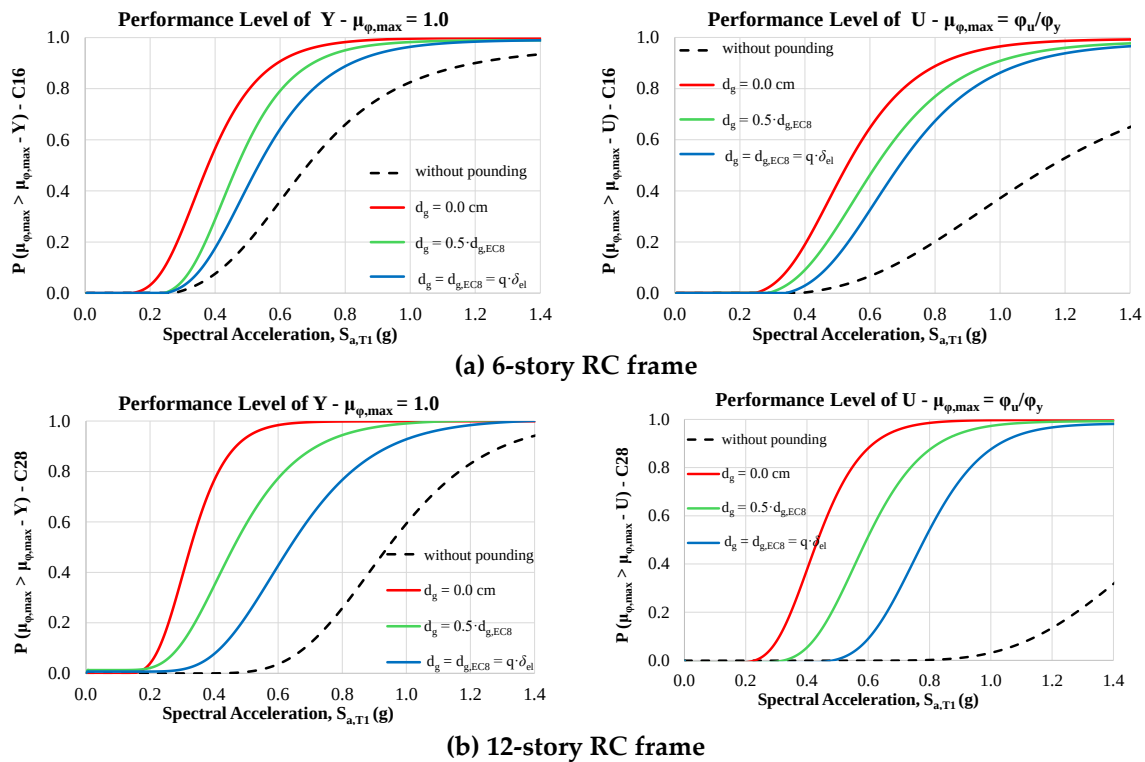


Figure 4. Fragility curves of $\mu_{\phi,max}$ at PLs, and at different gap distances (d_g) for the multistory RC structures subjected to floor-to-floor pounding; (a) 6-story RC frame, and (b) 12-story RC frame. Results for (i) $d_g = 0.0$ cm, (ii) $d_g = 0.5d_{g,EC8}$, (iii) $d_g = d_{g,EC8}$, and iv) without pounding.

In Figure 2 the probability of pounding considering different separation gaps (d_g) is assessed based on the maximum displacements ($\delta_{\max}$) at the top contact point of the examined RC frames. These results indicate that when the separation distance d_g is increased, the probability of pounding decreases for the same value of $S_{a,T1}$. Also, while the probability of pounding is 100% when the structures are in contact from the beginning, in the case of free vibration the corresponding probability is zero.

On the other hand, the consequences of the structural pounding on the damage level of the structure can be quantified only if the performances of critical global and/or local EDPs are properly involved in the probabilistic methods. So, in this study, the vulnerability of the structures in terms of $IDR_{\max}$ (global EDP) and $\mu_{\phi,\max}$ (local EDP) is assessed and presented in Figures 3 and 4.

Figures 3 and 4 demonstrate that the fragility curves resulting from structural pounding configurations are shifted to lower values of $S_{a,T1}$ compared to the corresponding fragilities without pounding (free vibration of the structure). Also, as it was expected, the impact of the pounding on the vulnerability of the structures is increased as the initial gap distance d_g between the adjacent structures is decreased. Evaluating the seismic performance of the RC frames at different capacity levels, it can be stated that as the performance level becomes more demanding, the fragility curves shift towards higher values of $S_{a,T1}$.

Of course, these observations align with existing knowledge on the seismic performance of structures under pounding conditions using probabilistic procedures.

To further quantify the consequences of the structural pounding, the median values of spectral acceleration at the fundamental period T_1 of the structures ($S_{a,T1}$) are estimated based on the PSDMs of $IDR_{\max}|S_{a,T1}$ and $\mu_{\phi,\max}|S_{a,T1}$. These values depict the $S_{a,T1}$ at which the EDPs of $IDR_{\max}$ and $\mu_{\phi,\max}$ reach a specific performance (capacity) level. Hereafter, the median values of $S_{a,T1}$ are referred to as $S_{a,wp}$ when pounding conditions are precluded and as $S_{a,dg}$ in pounding conditions. So, Figure 5 shows the $S_{a,wp}$ of the RC frames. It can be observed (Figure 5a) that the $S_{a,wp}$ at which the 6-story RC frame reaches the capacity of $IDR_{\max}$ at any PL is greater than the corresponding values of $S_{a,wp}$ of the 12-story RC frame. The 6-story reaches the capacity of $IDR_{\max}$ at the performance level of IO ($IDR_{\max}$ -IO) with a magnitude of $S_{a,wp}$ about 1.5 times greater than that of the 12-story frame. In the case of $IDR_{\max}$ -CP, this increment is 1.3.

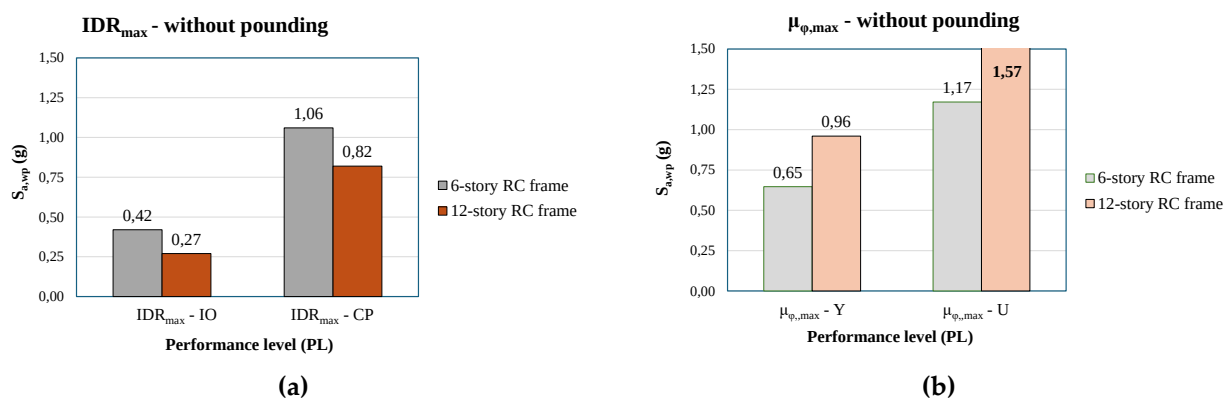


Figure 5. Median values of $S_{a,wp}$ for which $IDR_{\max}$ (a), and $\mu_{\phi,\max}$ (b) reach their capacity level at the examined PLs of each RC frame. Examined cases without structural pounding conditions.

Now, evaluating the performance based on the demands of $\mu_{\phi,\max}$, it can be observed (Figure 5b) that the 12-story RC frame is less vulnerable than the 6-story frame, as the examined column reaches its capacity level (Y or U) at higher values of $S_{a,wp}$. Nevertheless, the crucial seismic demand parameter of both structures is the $IDR_{\max}$, as lower values of $S_{a,wp}$ are required to reach the PLs compared to $\mu_{\phi,\max}$, even at the CP state. This condition is compatible with the seismic design philosophy. In all cases, the more demanding performance levels of $IDR_{\max}$ or $\mu_{\phi,\max}$ lead to increased values of $S_{a,wp}$.

Afterwards, in Figure 6, the median values of $S_{a,dg}$ at which the corresponding EDPs ($IDR_{\max}$ or $\mu_{\phi,\max}$) reach their ultimate capacity level are presented for the examined structural pounding cases.

It can be observed that the structures reveal similar performance characteristics compared to the case without pounding when the assessment is based on IDR_{max} . Indeed, the 6-story RC frame is again less vulnerable in IDR_{max} than the 12-story RC structure, as it reaches the capacity level of CP at greater values of $S_{a,dg}$. The same results hold for the capacity level of $IDR_{max} - IO$, and when the assessment is based on either $\mu_{\varphi,max} - Y$ or $\mu_{\varphi,max} - U$; the tall building (12-story RC frame) is more vulnerable than the 6-story frame.

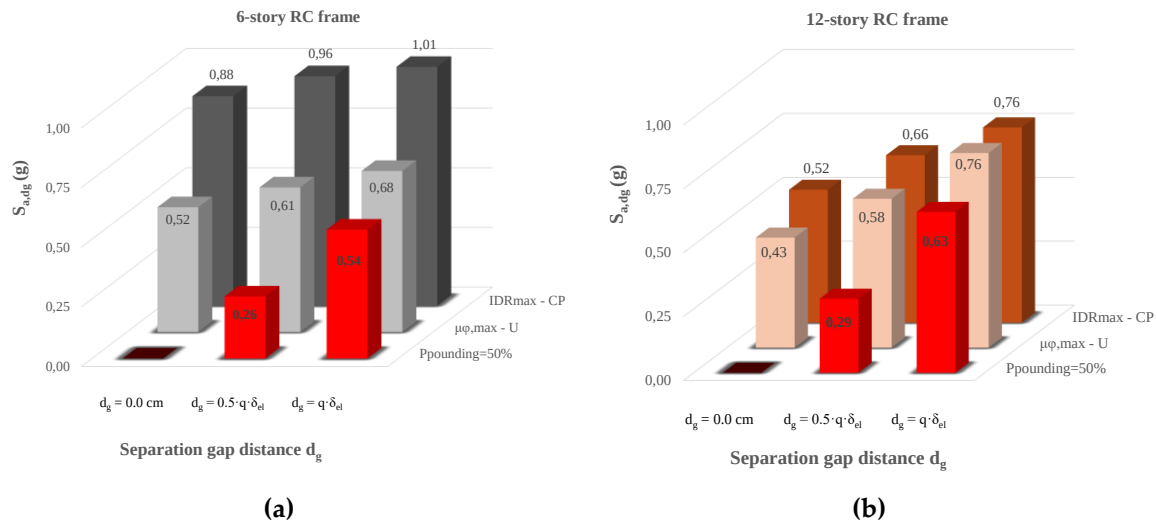


Figure 6. Median values of $S_{a,dg}$ as a function of d_g at $IDR_{max} - CP$, $\mu_{\varphi,max} - U$ and when the probability of pounding is equal to 50%.

The notable information herein is that these results (Figure 6) indicate that to ensure the safe performance of the structural systems under pounding conditions, the local demands of the critical columns should be considered. As shown in Figure 6 the structures reach the PL of $\mu_{\varphi,max}$ at a value of $S_{a,dg}$ that is lower than the $S_{a,dg}$ required to reach the examined PL of IDR_{max} . This reduction is more pronounced in the case of the 6-story RC frame and decreases as the building height increases. So, structural pounding significantly alters the seismic performance of structures compared to the case without pounding, making the ductility-based EDP, rather than the displacement-based EDP, the critical demand parameter for the evaluated RC frames.

Having evaluated the $S_{a,dg}$, the corresponding probability of pounding PL (referred hereafter as $P_{pounding}$) can also be defined through the fragilities of δ_{max} . In this way, the probability of pounding when a specific EDP-PL is achieved is directly expressed. These results are presented in Figure 7, where it can be observed that in most cases, the probability of pounding at the examined $S_{a,dg}$ -PL is greater than 50%, even when the gap distance provided by EC8 is applied (Figure 7). Meaning that the use of the δ_{max} ($P_{pounding}$) as a sole criterion to design and assess the structural pounding phenomenon may yield conservative solutions.

For example, in the case of the 6-story RC frame if we choose as an acceptable/targeting value of $P_{pounding}$ the 50% when (i) $d_g = 0.5d_{g,EC8}$ and (ii) $d_g = d_{g,EC8}$, then the corresponding acceptable values of $S_{a,T1}$ will be 0.26g and 0.54g, respectively (see Figure 6). However, these values are less than the one deduced based on the performance level of either IDR_{max} or $\mu_{\varphi,max}$ at the collapse limit states. As shown in Figure 6a the minimum acceptable $S_{a,dg}$ among IDR_{max} -CP and $\mu_{\varphi,max} - U$ is 0.61g when (i) $d_g = 0.5d_{g,EC8}$ and 0.68g when (ii) $d_g = d_{g,EC8}$, respectively. The same results were noted for the 12-story RC frame (see Figure 6b).

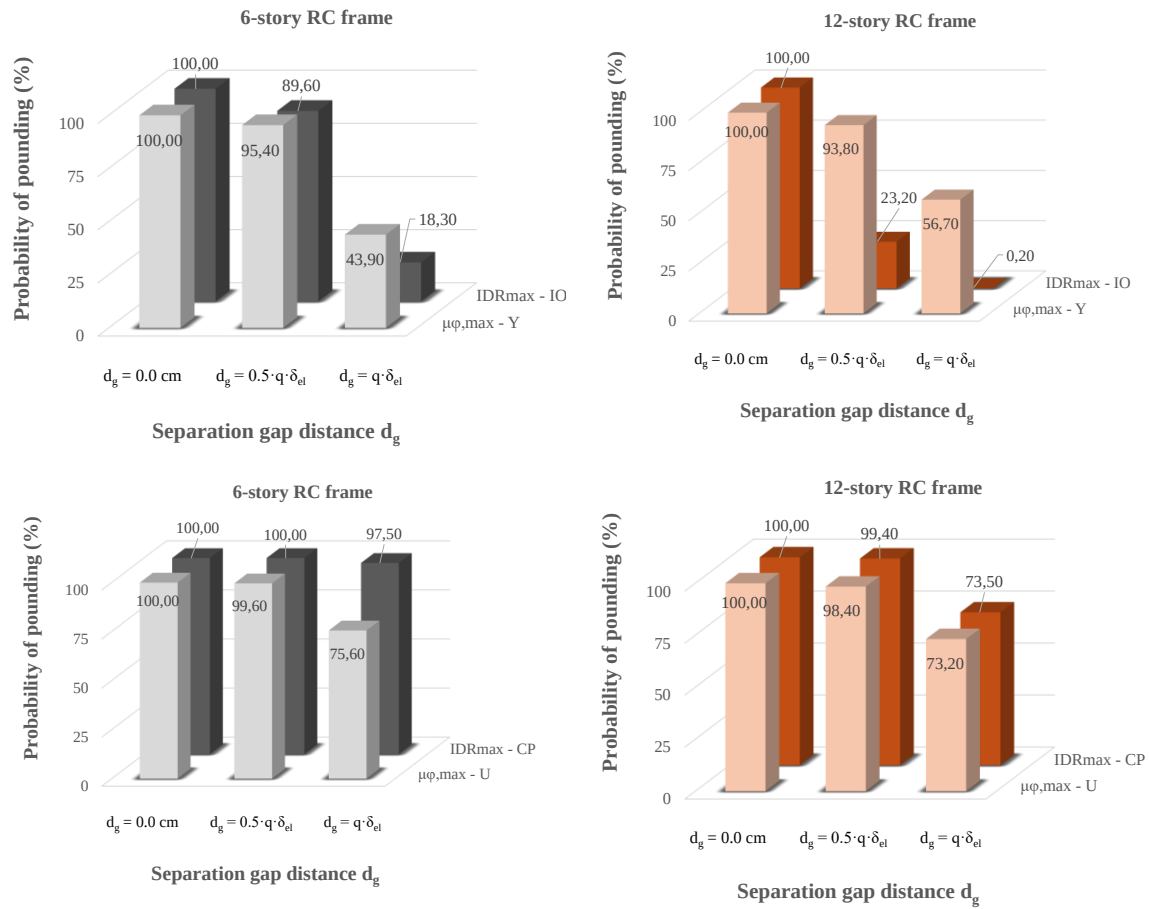
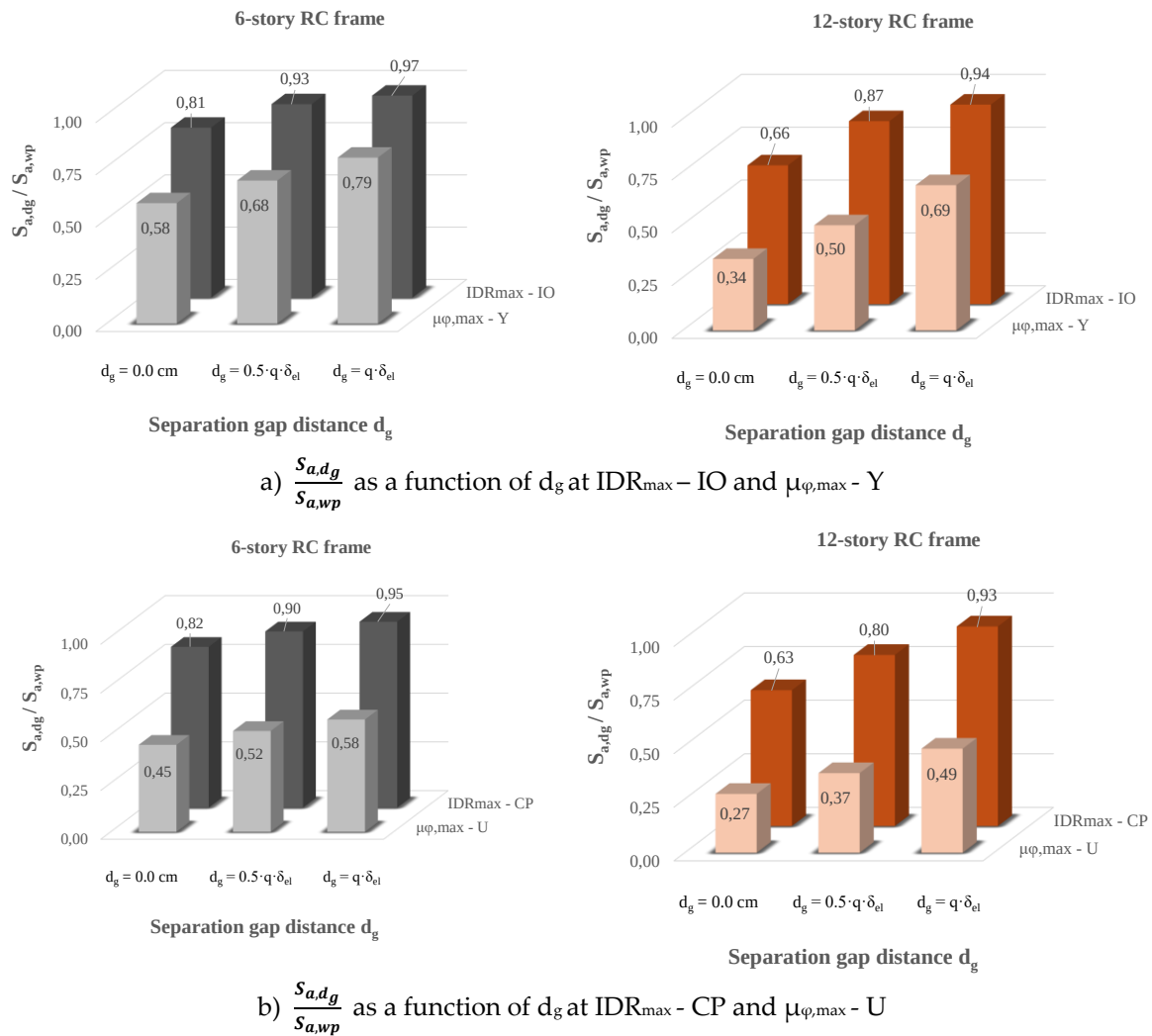


Figure 7. Probability of pounding as a function of d_g at different performance levels of $IDR_{\max}$, and $\mu\phi, \max$. Results for (i) $d_g=0.0 \text{ cm}$, (ii) $d_g=0.5d_{g,EC8}$, and (iii) $d_g=d_{g,EC8}$.

Finally, to further quantify the pounding impact on the values of $S_{a,wp}$ the ratio of $\frac{S_{a,dg}}{S_{a,wp}}$ is examined. These results are presented in Figure 8 as a function of the pre-defined separation gap distances, d_g , for both displacement-based and ductility-based performance levels. It can be observed that structures are expected to reach the examined capacity level (PL) at an earthquake intensity level less than the expected one in the case of no pounding, since $\frac{S_{a,dg}}{S_{a,wp}} < 1.0$ in all the examined cases of this study. The observed reduction is more intense when the $\mu\phi, \max$ - PLs of the columns are assessed.

Also, an increase of stories from 6 to 12 decreases the ratio of $\frac{S_{a,dg}}{S_{a,wp}}$. Therefore, the most critical pounding cases are posed on the tall building (12-story RC frame). This observation holds whether the assessment is based on the $IDR_{\max}$ - PLs or the $\mu\phi, \max$ - PLs.



$S_{a,dg}$ is the median intensity level $S_{a,T1}$ at which an EDP of the structure reaches its capacity level at the examined PL, when structural pounding conditions and the separation gap distance are involved.

$S_{a,wp}$ is the median intensity level $S_{a,T1}$ at which an EDP of the structure reaches its capacity level at the examined PL, without considering structural pounding effects.

Figure 8. Evaluation of the examined critical EDPs to assess an acceptable $S_{a,dg}$ considering structural pounding conditions at different PLs, and at different separation gap distances; (i) $d_g = 0.0$ cm, (ii) $d_g = 0.5d_{g,EC8}$, and (iii) $d_g = d_{g,EC8}$.

5. Conclusions

The main objective of this study is to provide insight into the importance of including critical EDPs to predict and control the severe damage caused by seismic pounding between adjacent multistory reinforced concrete buildings. Displacement-based and ductility-based EDPs are evaluated at different performance levels and at different separation distances d_g .

The seismic performance of 6-story and 12-story RC frame structures is assessed based on probabilistic methods. The structures are subjected to floor-floor pounding caused by an adjacent shorter and stiffer structure, with the top contact point located at the middle of the building's total height. Nonlinear dynamic analyses are carried out (IDA method) and PSDMs in terms of $\delta_{max} | S_{a,T1}$, $IDR_{max} | S_{a,T1}$, and $\mu\phi_{max} | S_{a,T1}$ are developed. Then fragility curves of the examined RC frames at different performance levels are defined, considering a) $d_g = d_{g,EC8} = q \cdot \delta_{el}$ (Eurocode's requirement), b) $d_g = 0.5d_{g,EC8}$, c) $d_g = 0.0$ cm (structures are in contact from the beginning), and d) free vibration of the structures (case without structural pounding effect). Then, the median value of $S_{a,T1}$ that

corresponds to an acceptable capacity level (acceptable PL) of an EDP is evaluated, and the probability of earthquake-induced pounding between adjacent structures is properly involved.

The results of this study deduce the following remarks:

- The first observations align with existing knowledge on the seismic performance of structures under pounding conditions using probabilistic procedures. Indeed, fragility curves due to structural pounding configurations are shifted to lower values of $S_{a,T1}$ compared to the corresponding fragilities without pounding (free vibration of the structure). Also, as it was expected, the impact of the pounding on the vulnerability of the structures is increased as the initial gap distance d_g between the adjacent structures is decreased. Evaluating the seismic performance of RC frames at different capacity levels, it can be stated that as the performance level becomes more demanding, the fragility curves shift towards higher values of $S_{a,T1}$.
- Structural pounding significantly alters the seismic performance of structures compared to the case without pounding, making the ductility-based EDP, rather than the displacement-based EDP, the critical demand parameter for the evaluated RC frames.
- The use of the δ_{max} ($P_{pounding}$) as a sole criterion to design and assess the structural pounding phenomenon yields conservative solutions.
- The structures reach their capacity level (examined PL) at an earthquake intensity level less than the expected one in the case of no pounding (in all the examined cases $\frac{S_{a,dg}}{S_{a,wp}} < 1.0$).
- An increase of stories from 6 to 12 decrease the ratio of $\frac{S_{a,dg}}{S_{a,wp}}$. Therefore, the most critical pounding cases are posed on the 12-story RC frames.
- The probability of pounding cannot be set to zero even when the gap distance provided by EC8 is applied.
- Excluding structural pounding consequences from the probabilistic frameworks may lead to unsafe performance-based seismic assessment or design provisions regarding the seismic risk of colliding buildings.

Author Contributions: Conceptualization, M.J.F. and E.G.T.; Methodology, M.J.F. and E.G.T.; Formal analysis, M.J.F. and E.G.T.; Writing—original draft, M.J.F. and E.G.T.; Writing—review & editing, M.J.F. and E.G.T. All authors have read and agreed to the published version of the manuscript.

Funding: This research received no external funding.

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: The data presented in this study are available on request from the corresponding author.

Conflicts of Interest: The authors declare no conflict of interest.

References

1. Favvata, M.J. Minimum required separation gap for adjacent RC frames with potential inter-story seismic pounding. *Eng. Struct.* 2017, 152, 643–659.
2. Tubaldi, E.; Freddi, F.; Barbato, M. Probabilistic seismic demand model for pounding risk assessment. *Earthq. Eng. Struct. Dyn.* 2016, 45, 1743–1758.
3. Nazri, M.; Miari, M.A.; Kassem, M.M.; Tan, C.G.; Farsangi, E.N. Probabilistic evaluation of structural pounding between adjacent buildings subjected to repeated seismic excitations. *Arab. J. Sci. Eng.* 2018, 44, 4931–4945.
4. Flenga, M.G.; Favvata, M.J. Fragility curves and probabilistic seismic demand models on the seismic assessment of RC frames subjected to structural pounding. *Appl. Sci.* 2021, 11, 8253.

5. Flegga, M.G.; Favvata, M.J. Global and local performance levels on the probabilistic evaluation of the structural pounding effect between adjacent RC structures. In Proceedings of the 11th International Conference on Structural Dynamics (EURODYN 2020), Athens, Greece, 2020; Volume 2, pp. 3762–3779.
6. Flenga, M.G.; Favvata, M.J. Fragility assessment of the inter-story pounding risk between adjacent reinforced concrete structures based on probabilistic seismic demand models. In Proceedings of the 13th International Conference on Earthquake Resistant Engineering Structures (ERES 2021), Online Conference, 2021; pp. 3–13.
7. Flenga, M.G.; Favvata, M.J. Probabilistic seismic assessment of the pounding risk based on the local demands of a multistory RC frame structure. *Eng. Struct.* 2021, 245, 112789.
8. Flenga, M.G.; Favvata, M.J. Investigating the efficiency and the sufficiency of IMs for the probabilistic seismic assessment of the pounding effect between adjacent RC structures. In Proceedings of the 9th ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering (COMPDYN 2023), Athens, Greece, 12–14 June 2023.
9. Flenga, M.G.; Favvata, M.J. The effect of magnitude M_w and distance R_{rup} on the fragility assessment of a multistory RC frame due to earthquake-induced structural pounding. *Buildings* 2023, 13, 1832.
10. Mohamed, H.; Romão, X. Seismic fragility functions for non-seismically designed RC structures considering pounding effects. *Buildings* 2021, 11, 665.
11. Kazemi, F.; Miari, M.; Jankowski, R. Investigating the effects of structural pounding on the seismic performance of adjacent RC and steel MRFs. *Bull. Earthq. Eng.* 2021, 19, 317–343.
12. Miari, M.; Jankowski, R. Incremental dynamic analysis and fragility assessment of buildings founded on different soil types experiencing structural pounding during earthquakes. *Eng. Struct.* 2022, 252, 113118.
13. Sinha, R.; Rao, B.N. Seismic fragility of non-ductile RC frames for pounding risk assessment. *Structures* 2023, 56, 104865.
14. Kazemi, F.; Asgarkhani, N.; Manguri, A.; Jankowski, R. Seismic probabilistic assessment of steel and reinforced concrete structures including earthquake-induced pounding. *Arch. Civ. Mech. Eng.* 2024, 24, 178.
15. Eurocode 2: Design of Concrete Structures. Part 1-1: General Rules and Rules for Buildings; EN 1992-1-1; European Committee for Standardization: Brussels, Belgium, 2002.
16. Eurocode 8: Design of Structures for Earthquake Resistance. Part 1: General Rules, Seismic Actions and Rules for Buildings; EN 1998-1; European Committee for Standardization: Brussels, Belgium, 2004.
17. Karayannis, C.G.; Favvata, M.J. Earthquake-induced interaction between adjacent reinforced concrete structures with non-equal heights. *Earthq. Eng. Struct. Dyn.* 2005, 34, 1–20.
18. Applied Technology Council (ATC). Seismic Evaluation and Retrofit of Concrete Buildings; Report No. ATC-40; Applied Technology Council: Redwood City, CA, USA, 1996.
19. Vamvatsikos, D.; Cornell, C.A. Incremental dynamic analysis. *Earthq. Eng. Struct. Dyn.* 2002, 31, 491–514. <https://doi.org/10.1002/eqe.141>.
20. PEER Ground Motion Database. Available online: http://peer.berkeley.edu/peer_ground_motion_database (accessed on 10 February 2017)

Disclaimer/Publisher's Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.