# Influence of bi-directional seismic pounding on the inelastic demand distribution of three adjacent multi-storey R/C buildings

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**Abstract.**   Interaction between closely-spaced buildings subject to earthquake induced strong ground motions, termed in the literature as "seismic pounding", occurs commonly during major seismic events in contemporary congested urban environments. Seismic pounding is not taken into account by current codes of practice and is rarely considered in practice at the design stage of new buildings constructed "in contact" with existing ones. Thus far, limited research work has been devoted to quantify the influence of slab-to-slab pounding on the inelastic seismic demands at critical locations of structural members in adjacent structures that are not aligned in series. In this respect, this paper considers a typical case study of a "new" reinforced concrete (R/C) EC8-compliant, torsionally sensitive, 7-story corner building constructed within a block, in bi-lateral contact with two existing R/C 5-story structures with same height floors. A non-linear local plasticity numerical model is developed and a series of non-linear time-history analyses is undertaken considering the corner building "in isolation" from the existing ones (no-pounding case), and in combination with the existing ones (pounding case). Numerical results are reported in terms of averages of ratios of peak inelastic rotation demands at all structural elements (beams, columns, shear walls) at each storey. It is shown that seismic pounding reduces on average the inelastic demands of the structural members at the lower floors of the 7-story building. However, the discrepancy in structural response of the entire block due to torsion-induced, bi-directionally seismic pounding is substantial as a result of the complex nonlinear dynamics of the coupled building block system.

**Keywords:**   seismic pounding; EC8 compliant buildings; three-dimensional model; ductility demand; spectrum compatible accelerograms; incremental dynamic analysis

## 1. Introduction

Earthquake-induced strong ground motions may cause interaction (seismic pounding), between closely-spaced vibrating buildings in contemporary congested urban environments. The occurrence and severity of seismic pounding between adjacent buildings depend on the clearance (separation gap) between the structures, on their geometry and dynamic properties, and on the

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indicate that seismic pounding occurs commonly in metropolitan areas and may significantly contribute to the overall economic and human losses due to earthquakes (EERI 1994, Anagnostopoulos 1994, 1996, Kasai & Masai 1997, Cole *et al.* 2012). Such observations triggered the interest of the research community to study the response of seismically excited adjacent buildings interacting and pounding due to insufficient clearance. Specifically, following early works on the topic (Anagnostopoulos 1998, Wolf & Skrikerud 1980), considerable research work has been devoted in recent years to develop practically useful analytical and computational models for the reliable representation of pounding forces (Anagnostopoulos 2004, Jankowski 2005, Muthukumar and DesRoches 2006, Dimitrakopoulos *et al.* 2009, Ye *et al.* 2009, Cole *et al.* 2011) to study the effects of these forces to the dynamic response and to the structural integrity of various types of colliding structures (Mouzakis and Papadrakakis 2004, Jankowski 2008, 2009, 2012, Mahmoud and Jankowski 2010, Polycarpou and Komodromos 2010, Giaralis and Spanos 2011), as well as to propose strategies to mitigate these effects (Spiliopoulos and Anagnostopoulos 1996, Xu *et al.* 1999, Anagnostopoulos and Karamaneas 2008, Lopez-Garcia and Soong 2009, Polycarpou and Komodromos 2011).

The current consensus suggests that seismic pounding between adjacent structures whose floor diaphragms lie at different levels, result in slab-to-column collisions and may induce severe local, primarily shear, damages to the columns. Such localized damages contribute significantly to the overall earthquake-induced damage at the system level and depending on their localization, may even trigger progressive, global collapse (Jeng and Tzeng 2000, Karayannis and Favvata 2005, Anagnostopoulos and Karamaneas 2008). However, in the case of adjacent buildings with equal storey floor levels, seismic pounding involves slab-to-slab collisions and, thus, no local loss of stiffness and/or strength to the lateral force resisting structural system takes place. In such cases, the influence of seismic pounding to the global response of structures becomes the issue of concern.

In this context, a parametric study was undertaken in Jankowski (2008) to investigate the influence of slab-to-slab pounding to the seismic response of two 3-storey, double-symmetric in plan, frame buildings considering material non-linearity. The structures were simultaneously subject to the three components of the strong ground motion associated with a specific historical earthquake record and results on the influence of pounding effects with regard to the clearance between the structures, their yielding strength and their inertial and stiffness properties have been reported. The main conclusion was that pounding is more critical for the structure with the lower mass. This conclusion has been further confirmed in Jankowski (2009) who considered, through a detailed three-dimensional finite element (FE) analyses, the interaction and pounding of a reinforced concrete (R/C) building with its significantly lighter, attached, staircase tower of the same total height.

Another historical study of pounding involving under-designed masonry and R/C buildings is reported in Fiore and Monaco (2010). More recently, the influence of pounding to a multi-storey wood frame building located at the corner of a typical building block in San Francisco has been assessed within a probabilistic seismic performance-based framework (Maison *et al.* 2012). The above conspectus of recent published work reveals that research efforts to assess the influence of seismic pounding have focused either on simplified "academic examples" of structures represented by two-dimensional (2D) or three-dimensional (3D) FE models or on real-life case-studies of under-designed buildings. Still though, the common case of high, newly designed buildings that are constructed in simultaneous, bi-lateral contact to a number of typically lower and under-designed buildings has not yet been thoroughly studied.

This is a very common problem nowadays in modern cities and, in fact, it is not yet taken into account by current codes of practice which only prescribe a minimum separation gap between the constructed building and its immediate built environment (CEN 2004a). This gap is usually determined based on response spectrum analysis in which the expected non-linear behavior of structures is only implicitly accounted for, through the behavior (or force reduction) factor. Field observations have shown that such considerations may not prevent seismic pounding in case of events less frequent than the design earthquake, while it is quite common that "as-built" structures may often have insufficient or no clearance at all for practical reasons.

It is noted that in most real-life cases, pounding of adjacent buildings takes place in a rather complex manner for a number of additional reasons (Jeng and Tzeng 2000, Maison *et al.* 2012) such as that: (i) buildings are not constructed in series but within blocks, hence, particularly the corner buildings are subject to bi-lateral pounding, and (ii) due to the lack of available space and the cost of land in modern cities, newer structures are typically higher and slender than older ones, a fact that is commonly associated with the significant contribution of their higher, primarily torsional, modes of vibration.

In this regard, this paper considers the case of a newly designed, 7-storey, reinforced concrete (R/C) building located at the corner of a block in a major metropolitan area, in contact with two adjacent, under-designed, 5-storey buildings. The condition is that for constructional purposes, the first building (hereafter denoted as "K") is in immediate contact with the other two (identified as "K1" and "K2"), thus, there is practically no separation gap. The assumption is also made that storey levels are at equal heights and that there is no shear slab penetration to the columns in contact (i.e. local damage is only attributed to slab-to-slab pounding).

Along these lines, the paper presents the development of a 3-Dimentional (3-D) finite element model of the three buildings comprising the block, as a means to comparatively assess the induced seismic damage in terms of rotational ductility demand at a local and system level, with and without pounding, both under the design earthquake and more severe seismic actions. An overview of the case studied, as well as details on the non-linear FE models developed, the incremental dynamic analysis framework adopted and the inelastic demands at various critical cross-sections of the considered structures is provided in the following sections.

## 2. Overview of the building block studied

### 2.1 Design considerations

The herein considered case study, though not identical, is based on a real building block of three adjacent, multi-storey R/C buildings bi-laterally interacting as shown in Fig. 1. The corner building "K" is assumed to be designed according to the European structural design code framework, that is, Eurocode 2 for R/C buildings (CEN 2004b) in conjunction with Eurocode 8 for earthquake resistant design (CEN 2004a) and the Greek National Annex, for a (design) spectrum assuming peak ground acceleration (PGA) of 0.16g, soil type "B", ductility class high (DCH) and behavior factor "q" equal to 3.0 (CEN 2004a). Concrete grade is taken as C20/25 (compressive strength equal to 20N/mm$^2$) and steel grade as S500 (yielding strength 500MPa). The modulus of elasticity of the reinforced concrete is taken equal to 29 GPa and its density is 25kN/m$^3$.

The side buildings "K1" and "K2" are assumed to be designed to the older version of the seismic code of 1985 corresponding to a triangularly distributed horizontal load equal to 8% of the

characteristics of the ground motion. In fact, field observations in the aftermath of major historical seismic events building weight and basic capacity design and ductility considerations. The height of the typical storey is equal to 3m, however, as seen in Fig. 1, the corner building "K" has a ground floor pilotis. Dead loads refer to self-weight of the reinforced concrete members plus 2,00kN/m2 for finishing, while live loads of 2,00kN/m2 are prescribed for all indoor slabs and 5,00kN/m2 are imposed to the balconies.

## 2.2 Finite element (FE) modeling assumptions

Three distinct FE models have been developed to scrutinize the effect of seismic pounding, i.e., one for "contact-free", individual, buildings "K", "K1", "K2", and a fourth FE model for the


(a) Typical floor plan


(b) Location of potential pounding at each floor


(c) Building block used as the basis for the case study considered


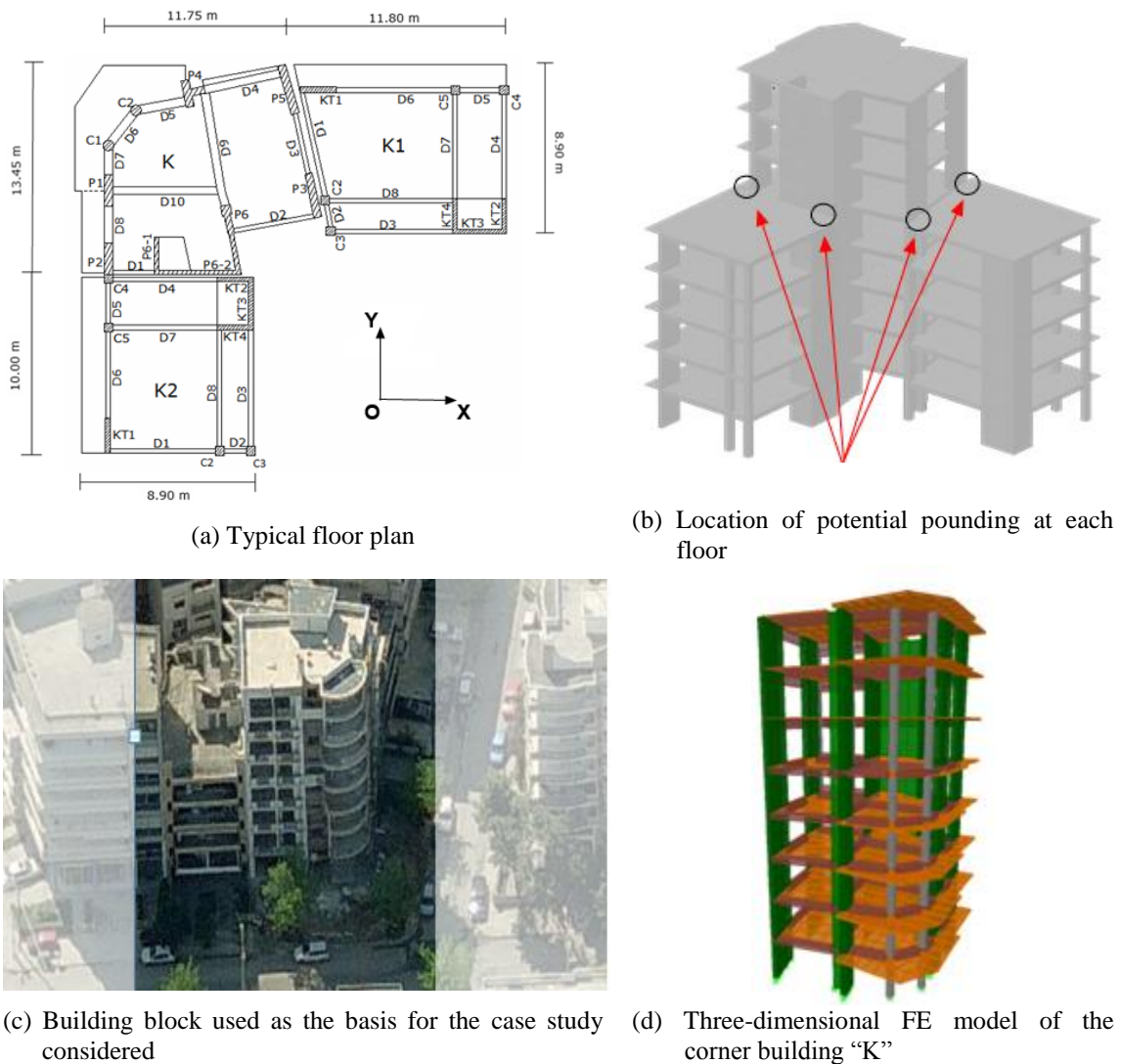(d) Three-dimensional FE model of the corner building "K"

Fig. 1 Considered three building complex

entire interacting complex as shown in Fig. 1. The commercial FE software SAP2000® (CSI 2012) has been used for all linear and non-linear analyses. Two-dimensional (2-D) quadrilateral shell elements were used to model slabs and shear walls, while beams and columns were modeled using linear 1-D frame elements (Fig. 2(a)).

Table 1 Natural periods of the considered buildings

| Building | 1$^{st}$ Natural period | 2$^{nd}$ Natural period | 3$^{rd}$ Natural period |
|---|---|---|---|
| K | 0.72s (Dominantly translational along x-x axis) $U_x= 50\%$ | 0.58s (Dominantly translational along y-y axis) $U_y= 70\%$ | 0.30s (Dominantly rotational) $R_z= 71.5\%$ |
| K1 | 0.65s (Rotational ) | 0.43s (Translational) | 0.21s (Rotational) |
| K2 | 0.58s (Rotational) | 0.41s (Translational) | 0.21s (Rotational) |



(a) Modeling of shear walls and cores using 2-D elements

(b) Modeling of shear walls and cores using 1-D elements
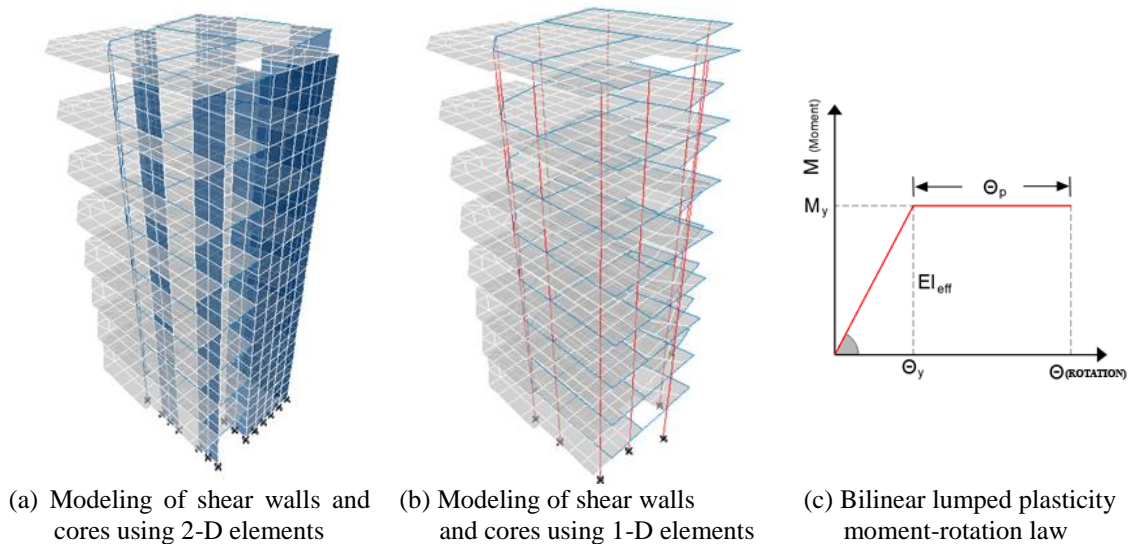
(c) Bilinear lumped plasticity moment-rotation law

Fig. 2 Finite element modeling assumptions

Table 1 reports the first three natural periods of the three buildings considered along with a qualitative description of the corresponding mode shapes obtained by means of a standard modal analysis to the models for fixed based conditions. It is noted that all structures have a significant torsional mode.

### 2.2.1 Material nonlinearity

Inelastic material behavior in flexure at all critical cross-sections of beams and columns is introduced by assuming lumped plasticity through rotational spring elements assigned at both ends of each frame element. A bilinear perfectly elasto-plastic moment–rotation (M-θ) relationship is assumed for each plastic hinge as shown in Fig. 2(c) after appropriate computation of the

corresponding moment-curvature (M-$\varphi$) relationships by means of standard fibre analysis with the program RCCOLA (Kappos 1993). The plastic rotation $\theta_p$ is computed by the equation (Priestley *et al.* 1996) as follows

$$\theta_p = L_p \left( \varphi_u - \varphi_y \right)$$ (1)

where $\varphi_u$ and $\varphi_y$ are the ultimate and yielding curvatures, respectively, determined from fibre analysis and   plastic hinge length $L_p$ is given by

$$L_p = 0.08L + 0.022 f_y d \ .$$ (2)

In the above equation, $L$ the distance from the critical section of the plastic hinge to the point of contraflexure, $f_y$ is the assumed yielding stress of the longitudinal reinforcement bars, and $d$ is the radius of the longitudinal reinforcement bars. The yield rotation $\theta_y$ is evaluated from the corresponding area in the curvature diagram, as $\theta=\int\varphi dx$, although the above procedure has been found to underestimate the actual $\theta_y$. In fact, the slope of the second branch of the M-$\theta$ curve is higher than that of the M-$\varphi$ curve and is dependent on the rotational ductility factor $\mu_\theta$ (Kappos & Sextos 2001). Nevertheless, the assumption is made that the yield rotation $\theta_y$ can be evaluated by the curvature diagram, thus, it is estimated as

$$\theta_y = 0.5\varphi_y L$$ (3)

Shear walls and concrete cores are modeled by means of an "equivalent central column" connected to the beams at the level of each floor using perfectly rigid virtual frame elements. This modeling strategy is necessary to allow for inelastic behavior at the base of shear walls and cores which is assumed as a critical cross-section in the earthquake resistant design of coupled R/C buildings. Bilinear rotational spring elements, defined in the same manner as detailed above for the case of beams and columns, are introduced at their base to account for the potential formation of plastic hinges. A typical topology of this modeling is juxtaposed with the FE model used in the design phase of the K building in Fig. 2(b) for the purpose of comparison. Special attention has been given to calibrate the model with the equivalent central columns to achieve similar modal properties with the FE models used in the design stage where shear walls and cores were explicitly modeled via 2-D shell elements (see e.g. Lew & Narov 1983).

### 2.2.2 Geometric nonlinearity
For the purposes of the present study, pounding is modeled using a uni-axial linear spring which is activated only under compression. To this aim, the built-in "gap" non-linear element of SAP2000 (CSI 2012) has been incorporated in the FE model combining all three buildings of the considered complex. Impact is assumed to take place at four locations at each floor level as shown in Fig. 1(b). Assuming that the buildings are initially in contact, which is also the case of the actual building block used as the reference for this study (Fig.1(c)), the pounding forces along the local longitudinal degree of freedom of each gap element can be expressed as follows

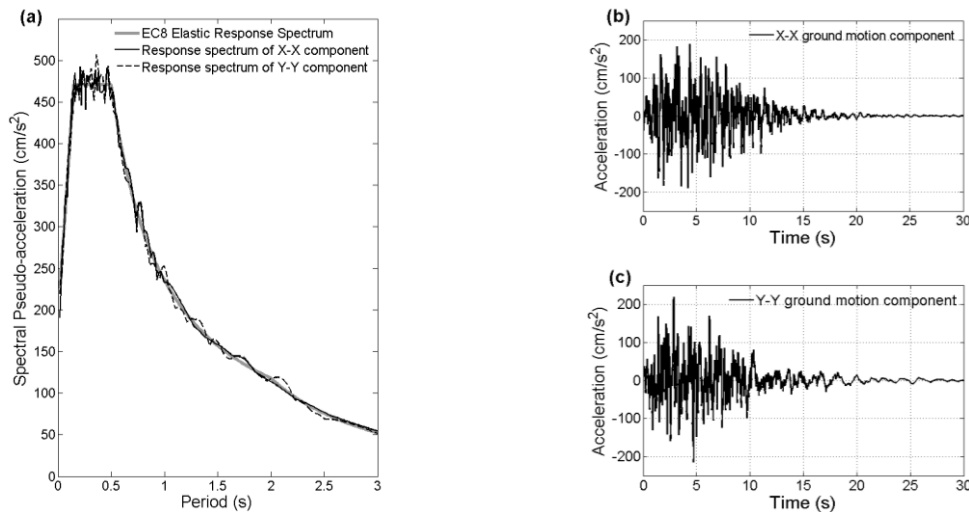$$f = \begin{cases} kx \, , x < 0 \\ 0 \, , x > 0 \end{cases}$$ (4)

where $k$ is the stiffness of the spring set equal to $10^7$ kN/m and $x$ is the relative displacement at the spring edges. It is noted that the adopted pounding model does not take into account contact

friction and local energy dissipation during pounding (Anagnostopoulos 2004, Mouzakis and Papadrakakis 2004, Jankowski 2005, Muthukumar and DesRoches 2006), hence, it is assumed that pounding does not contribute to the dissipation of the input seismic (kinetic) energy and, thus , it is inherently conservative in terms of peak response quantities. This is in alignment with the purposes of this study which seeks to "envelop" the pounding effect in terms of peak ductility demands following common earthquake resistance design considerations, rather than to explicitly represent and model in absolute terms the complicated phenomenon of seismic pounding.

## 3. Adopted incremental dynamic analysis framework

Earthquake ground motion is introduced through artificial accelerograms that are compatible with the Eurocode 8 response spectrum for the site of interest and are uniformly scaled for different levels of seismic intensity expressed in terms of PGA (i.e., 0.1≤PGA≤1.0g at a step of 0.1g), following an incremental dynamic analysis framework (Vamvatsikos and Cornell 2002). The progressive scaling permits the gradual yielding of the structure with increasing intensity and the investigation of the effect of bi-directional building pounding to the extent and location of the induced damage. It is noted that although the spectral acceleration at the natural period of a structure is a widely used intensity measure (IM), it is PGA that is adopted herein, since the particular study involves three coupled buildings for which the fundamental period is not common. It is also reported that ground motion variability is deliberately not taken into explicit consideration in this study in order to draw fundamental conclusions, based on the Eurocode 8 (uniform hazard) target response spectrum.

Two, equal intensity accelerograms, corresponding to the two principal directions of excitation (X-X, Y-Y), have been generated for each level of PGA using the wavelet-based stochastic approach detailed in (Giaralis and Spanos 2009) after close spectral matching along the entire



(a) Response spectra of accelerograms and target Eurocode 8 spectrum

(b) Time-histories of accelerograms

Fig. 3 Response spectra and time-histories of the Eurocode 8 compatible accelerograms

period range of interest (Fig. 3(a)). In particular, a Eurocode 8 response spectrum compatible non-stationary process is first derived (Giaralis and Spanos 2012). Next, two realizations of this process are generated and modified by means of  a  harmonic wavelet-based iterative procedure and a state-of-the-art baseline correction technique leading to very satisfactory spectral matching. The   time-histories of the thus obtained accelerograms are shown in Figs. 3(b) and 3(c).

Pertinent statistical attributes of the inelastic seismic demands to the horizontal (beams) and the vertical (columns and shear walls) members at every floor of each structure are monitored for various scaling factors of the input seismic action. To directly illustrate the effect of pounding on the damage induced at the three buildings, the rotational ductility demand $\mu_\theta$ at all distinct members of the three buildings is adopted as the principal Engineering Demand Parameter (EDP).

## 4. Effect of bi-directional pounding on the inelastic seismic demand distribution of the three coupled buildings

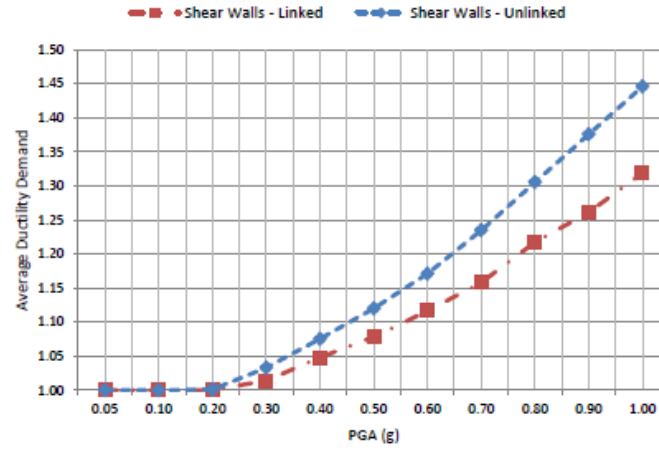### 4.1 Influence of strong ground motion severity

Following the development of the 3D, coupled finite element model of the three adjacent buildings comprising the block, a series of 10 non-linear time-history analyses are undertaken to probe into the dynamic response of both the bi-directionally interacting system and that of each individual building considered entirely uncoupled (i.e. as if the seismic joint was of infinite length). Then, seismic damage, expressed in terms of rotational ductility demand, is predicted using the adopted incremental dynamic analysis framework presented in section 3, for the case of the linked (coupled) and unlinked (uncoupled) buildings, K, K1, and K2.

Fig. 4 illustrates the variation of the average ductility demand at the base of the shear walls, the edge of the beams and the top and bottom of the columns at the ground floor of the seven-storey corner building "K", with and without pounding and for increasing seismic intensity (PGA). It is seen that independently of seismic pounding, structural damage at the shear walls and the beams of the ground storey is first initiated approximately at a peak ground acceleration of approximately 0.15g, a fact which is consistent with the capacity design of the "K" building (i.e., beam yielding precedes column failure) and the acceptance of damage for the design earthquake through the adoption of a behavior factor $q$=3.0.
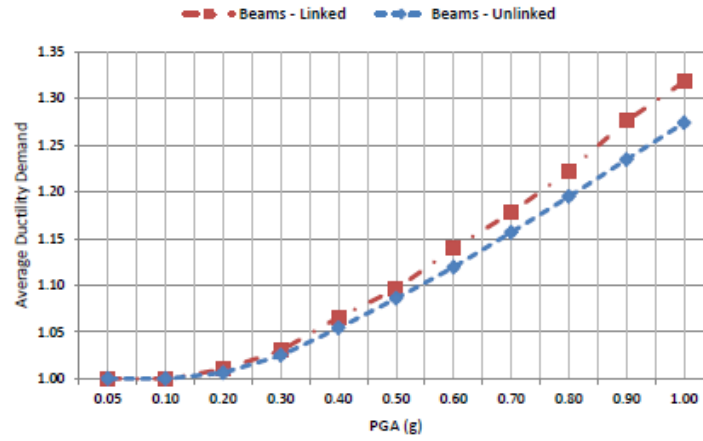
A second reasonable observation that is made is that, the effect of bi-directional pounding to the rotational ductility demand of the ground floor shear walls and beams is increasing with increasing intensity. Furthermore, shear walls of the corner "K" building seem to be relieved at the ground floor due to its multiple pounding with "K1" and "K2"; in particular, the average $\mu_\theta$ is reduced from 1.45 to 1.30 for the extreme case of PGA=1.0g. This is not the case though for beams which are critically affected by seismic pounding (Fig. 4 middle). This effect is even more profound in Fig. 5 where the detrimental influence on pounding to the beam damage is clearly seen at the 5th, 6th and 7th storey. In general, pounding effects do not significantly affect the seismic demands of the columns but this is primarily because seismic forces are resisted by the shear walls and the columns remain elastic even for high levels of PGA.

To better visualize the interaction between the three buildings due to seismic pounding, a series of additional illustrations is presented in Figs. 6-9 reporting the mean of the ductility demand ratio (i.e., E=$\mu_{\theta,linked}$/$\mu_{\theta,unlinked}$) as well as the standard deviation of this ratio of critical sections at each floor of all buildings, with and without pounding and for two different levels of ground motion
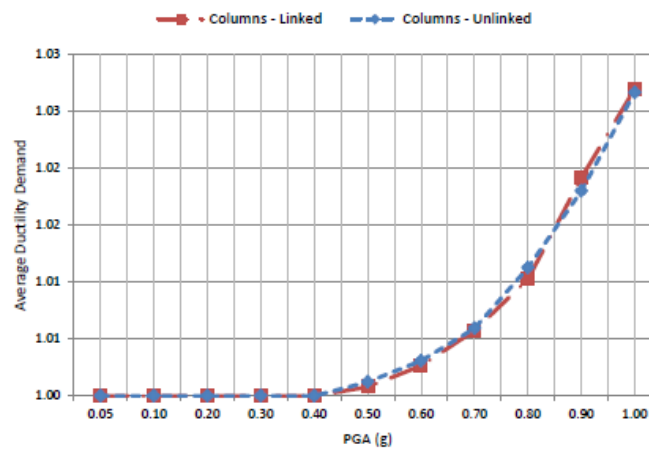
(a) Base of shear walls



(b) Edges of beams



(c) Edges of columns

Fig. 4 Average ductility demand of structural elements at the ground floor of the corner building "K", with pounding (linked) and without pounding (un-linked) for different levels of seismic intensity (PGA)
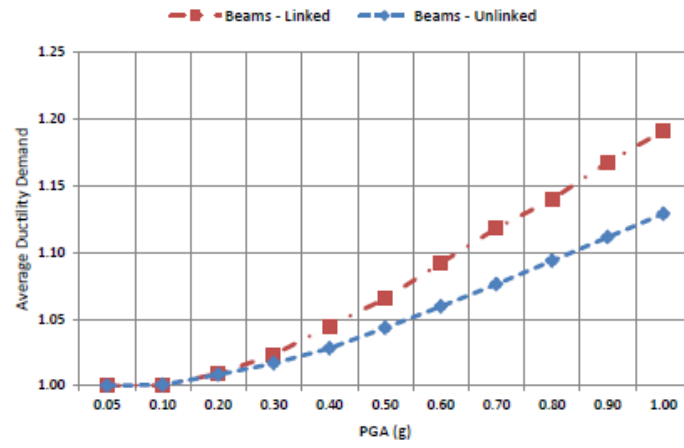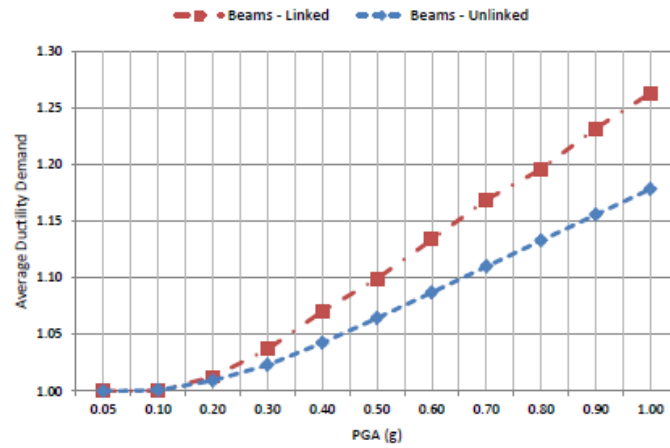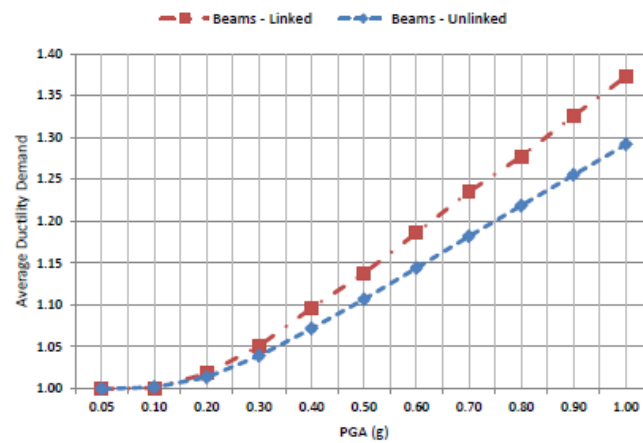
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(a) Edges of beams at 5$^{th}$ floor



(b) Edges of beams at 6$^{th}$ floor



(c) Edges of beams at 7$^{th}$ floor

Fig. 5 Average ductility demand of beam elements at various floors of the corner building "K", with pounding (linked) and without pounding (un-linked) for different levels of seismic intensity (PGA)

intensity (i.e. 0.5g and 0.9g).

Focusing again on the 7-storey corner building "K" it is also clearly seen that the vertical elements (Figs. 6 and 8) are generally either relieved *on average* (i.e., ground floor members independently of PGA) or show a negligible increase in ductility demand that does not exceed 1% (ratio E<1.01). It is critical to notice though, that this effect is only observed on average, while the significant variation of the demand in individual structural members is essentially suppressed. For instance, there are many cases where the $\mu+\sigma$ of the rotational ductility demand ratio ($\mu_{\theta,linked}/\mu_{\theta,unlinked}$) is almost doubled independently of the storey and the level of PGA examined. Similarly, the $\mu-\sigma$ of the rotational demand ration may well drop below 0.4. This is a clear indication of the significant effect of seismic pounding not only on absolute values of demand but particularly on the damage distribution, even in new buildings that are designed to modern seismic codes. The same observation is also valid for the beams of the corner building "K" where the discrepancy in ductility demand, with and without pounding, is indeed very high, even though *on average*, again, ductility demand is only increased by a mere 10%.

Studying the side, lower, buildings "K1" and "K2" the above remarks hold, as well. Again, on average, building "K1" is generally relieved (mean ratio E<1.00) in all structural members, in all stories and independently of ground motion intensity. However, the inelastic demand discrepancy remains substantial and there are numerous structural members where the local ductility demand is either doubled or dropped by more than 50% due to bi-directional pounding with the corner building "K". A close look at the result of building "K1" confirms once more the general trend of high structural response discrepancy.
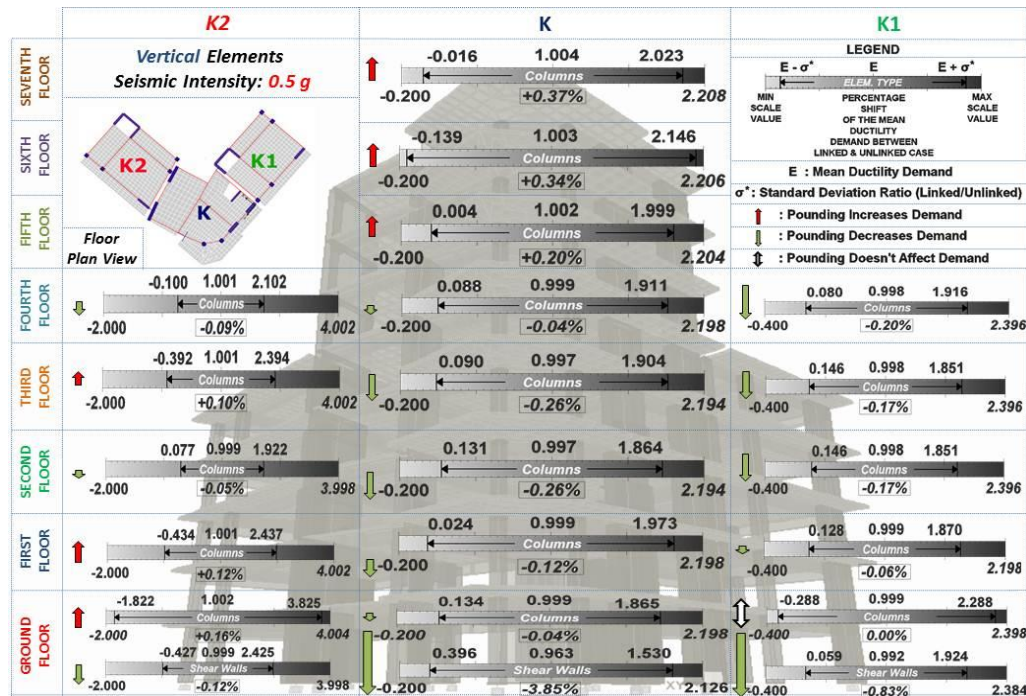


Fig. 6 Variation of inelastic demand (mean ± standard deviation) due to structural pounding to the vertical members at each floor for PGA= 0.5g

What is therefore seen from the herein reported numerical data is that, in contrast to the simpler cases studied in the literature, where buildings are aligned along a straight line and the lower buildings experience the most critical impact of seismic pounding, the dynamics of a coupled building block in full bi-directional contact is much more complex and difficult to predict. It is also seen that there is no clear trend which can be attributed to the different height of the buildings, since the torsional coupled behavior of the three interacting buildings can critically affect both the high-rise and the lower buildings simultaneously and to the same extent.

### 4.2 Influence of strong ground motion directionality

It can be deduced from Figs. 6-9 that, on average, pounding reduces ductility demands for all structural members and floors of the "K1" building, while it has a mixed effect for structural members of the of the "K2" building. Given that these two buildings are dominated by torsional response (see Table 1) and that they experience "single-sided" pounding, an additional series of non-linear time-history analysis following the same IDA framework as before have been performed to investigate the effect of directionality of the considered input ground motion. Specifically, a full set of results have been obtained having the strong ground motion component along X-X direction reversed. In Fig. 10 representative results for the "K1" building are presented indicating that the directivity of the strong ground motion affects considerably the seismic demands of single-sided pounding. In particular, reversing the direction of the X-X ground motion component imposes higher ductility demands for the "K1" building when pounding occurs. This result further reinforces the previous remark on the complexity of the effects of pounding in considering adjacent buildings interacting in 3-D within a complex building block.
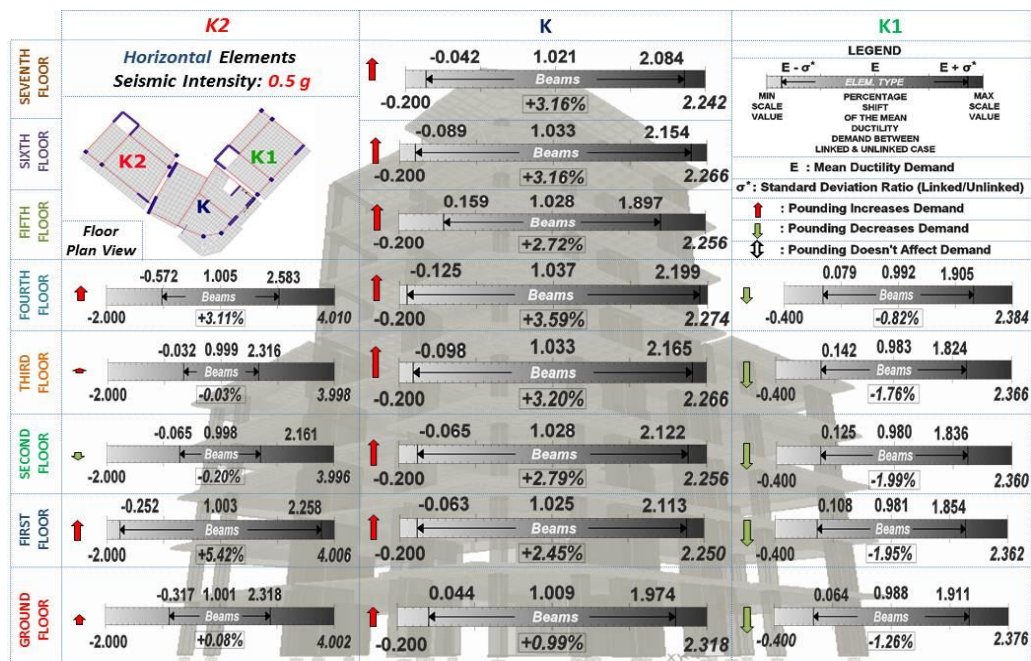


Fig. 7 Variation of inelastic demand (mean ± standard deviation) due to structural pounding to the beams at each floor for PGA= 0.5g
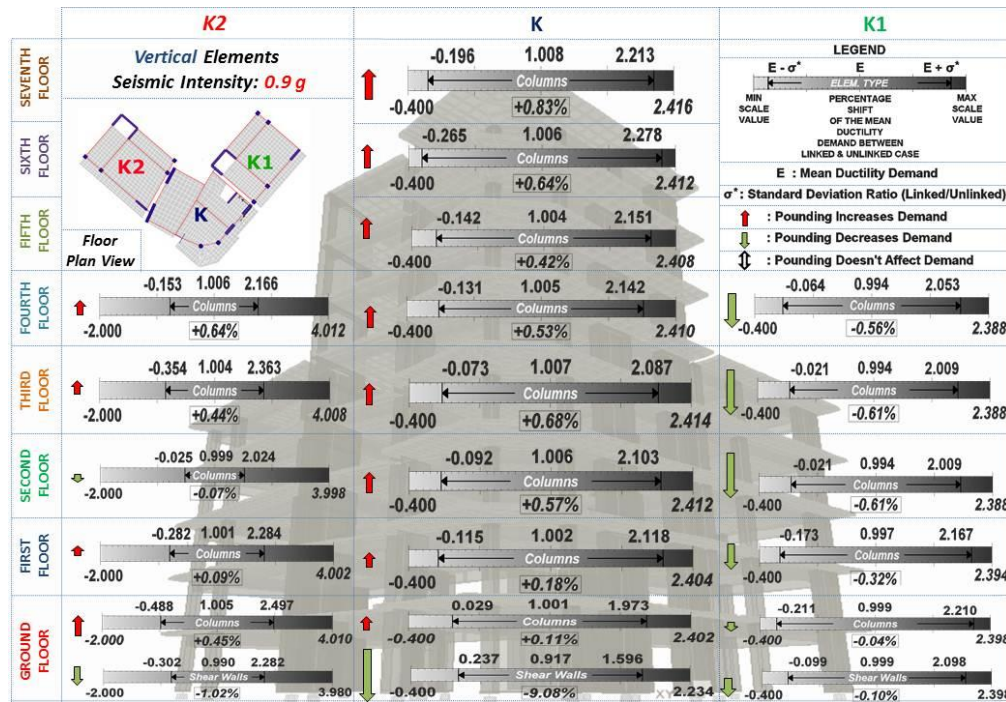
Fig. 8 Variation of inelastic demand (mean ± standard deviation) due to structural pounding to the vertical members at each floor for PGA= 0.9g
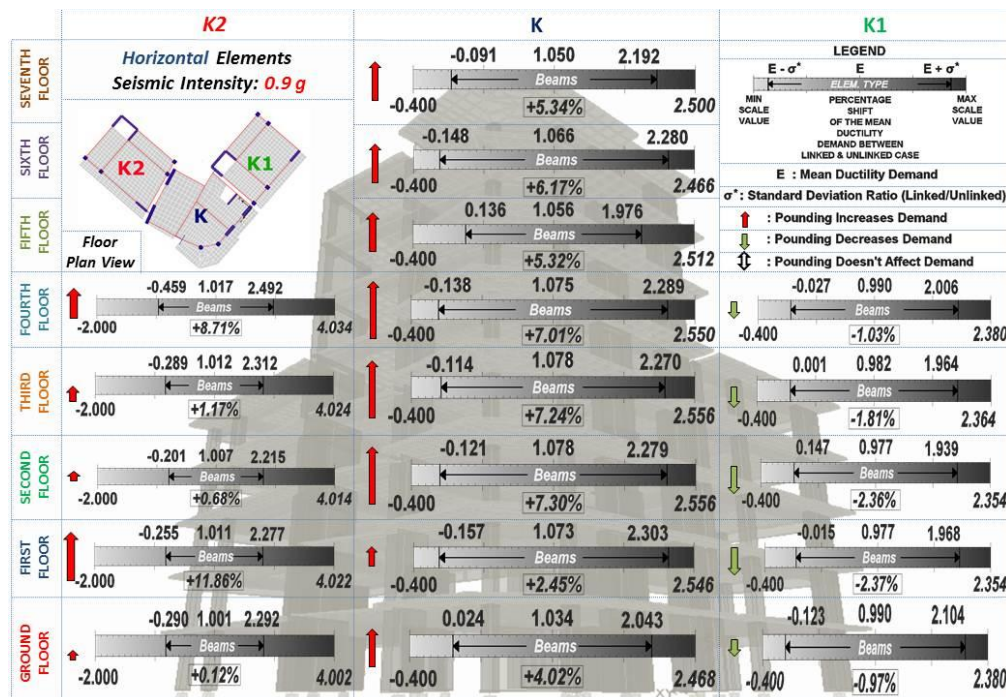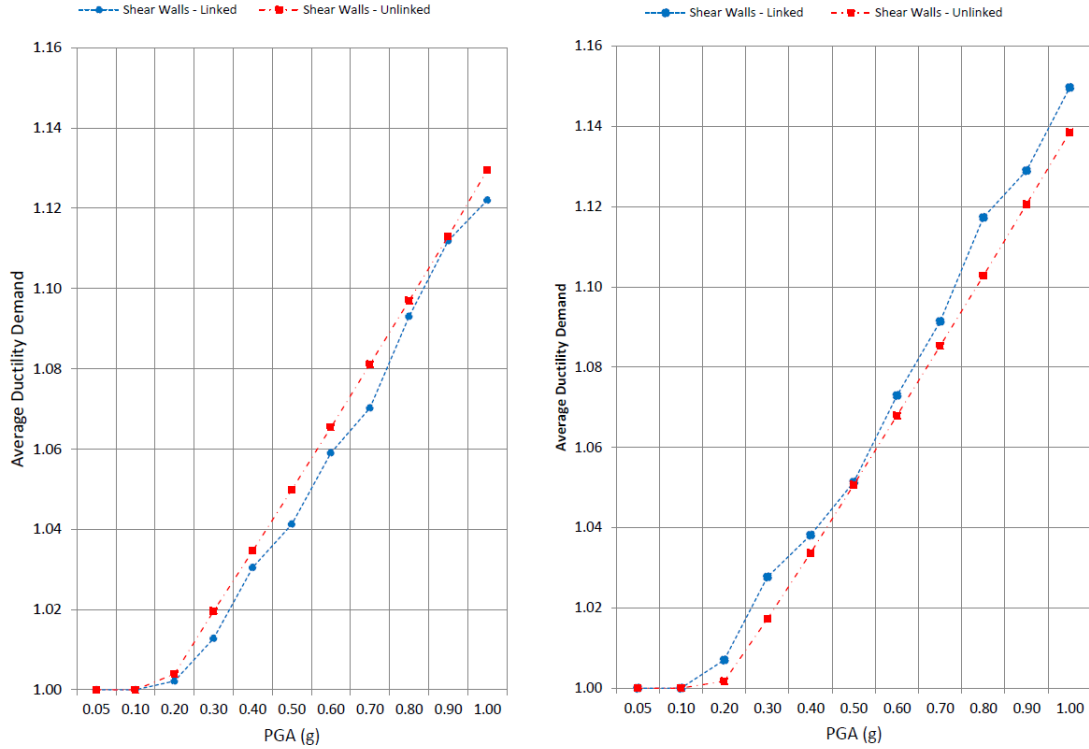


Fig. 9 Variation of inelastic demand (mean ± standard deviation) due to structural pounding to the beams at each floor for PGA= 0.9g

(a) X-X component of Fig. 3(b) acting along the "positive" O-X axis of Fig. 1(a)

(b) X-X component of Fig. 3(b) acting along the "negative" O-X axis of Fig. 1(a)

Fig. 10 Average ductility demand of shear walls at the ground floor of building "K1", with pounding (linked) and without pounding (un-linked) for different levels of seismic intensity (PGA)

## 5. Conclusions

A judicially chosen case study has been considered to illustrate the complex non-linear response of realistic building blocks, involving code-compliant R/C buildings which are (a) constructed in contact to under-designed, lower-rise, existing structures in metropolitan areas, (b) located at the corner of a building stock, and (c) are subject to bi-directional pounding due to torsion. Pertinent numerical data have been furnished to provide an insight as to what difference in terms of inelastic seismic demands (and consequently in terms of detailing) pounding would make in the design of new code-compliant R/C buildings.

Specifically, a detailed numerical model of the coupled 3D, interactive building block was developed and the inelastic demand distribution (expressed in terms of rotational ductility demand $\mu_\theta$) was computed for all members of all buildings, with and without pounding and for different levels of seismic intensity. These results demonstrate a general average trend of reduced inelastic demands of vertical structural members in the lower floors of the 7-story building and relatively higher demands in the upper stories when interaction between adjacent buildings takes place. The same is also seen on average for one of the two side buildings ("K1") which shows a minor decrease in inelastic demand of both beams and columns. What is important to notice though, is

that the discrepancy of the inelastic demand induced by seismic forces, with and without pounding, is significant: the mean plus one standard deviation of the ratio $\mu_{\theta,\text{linked}}/\mu_{\theta,\text{unlinked}}$ is greater than 2.0 almost in all cases of buildings and members examined. This is deemed to be interesting evidence that the trends observed in the literature with respect to the pounding of buildings aligned in series are not necessarily visible in the case of complex blocks of buildings colliding bi-directionally. Further research is warranted to account for the influence of record-to-record variability of the strong ground motion and of the seismic input directivity with respect to the individual building axes. Additional research is also needed in order to consider the premature shear failure of the under-designed, existing buildings.

## Acknowledgments

## Disclaimer

The case study presented was based on actual building geometries after appropriate adaptations that served the research objectives, hence, no specific assessment was made regarding the actual seismic performance of the considered buildings.

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