Probabilistic seismic demand and fragility assessment for evaluating the separation distance between adjacent buildings

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ABSTRACT: This study aims to develop a probabilistic seismic demand model (PSDM) for pounding risk assessment suitable for use within modern performance-based design frameworks. In developing a PSDM, different choices can be made regarding the intensity measures (*IM*s) to be used, the record selection, the analysis technique applied for estimating the system response for different *IM* levels, and the model to be employed for describing the response statistics given the *IM*. In the present paper, some of these choices are analyzed and discussed by considering the case of two adjacent buildings modeled as single-degree-of-freedom systems with linear and nonlinear hysteretic behavior. Based on the comparison, an optimal demand model is sought as the one that permits to achieve confident estimates of the response parameter of interest, i.e., the relative displacement demand, with few time-history analyses. This property allows reducing the complexity and computational cost associated with the pounding risk assessment.

1 INTRODUCTION

The seismic-induced pounding between adjacent buildings with inadequate separation distance is an undesirable event that can cause major damage and even structural collapse (Anagnostopoulos 1988, Cole et al. 2012). This issue is particularly relevant for structures located in metropolitan areas, due to limited availability of land space.

In order to minimize the pounding risk, design codes provide simplified procedures and rules for estimating the minimum separation distance avoiding pounding under a target seismic hazard scenario (BCJ 1997, ICBO 1997, TBC 1997, ECS 2005). However, these code procedures are characterized by unknown safety levels and, thus, do not permit to control explicitly the risk of pounding (Tubaldi et al. 2012). Recently, a reliability-based methodology has been proposed for the design of the separation distance between adjacent buildings which corresponds to a target probability of pounding during the design life of the buildings (Barbato and Tubaldi 2012). The methodology can be efficiently applied to the case of buildings modeled as linear systems, for which analytical technique can be employed to estimate with good accuracy the response statistics under the uncertain earthquake input. However, the application of the methodology to nonlinear building models is computationally demanding and further studies are required to improve its efficiency.

The objective of this paper is to advance the application of performance-based engineering concepts for the assessment of the pounding risk between adjacent buildings, and for the design of the separation distance corresponding to a target safety level. This objective is sought through the development of an efficient probabilistic seismic demand model (PSDM) (Padgett et al. 2008) for pounding risk assessment suitable for use within modern performance-based design frameworks such as the Pacific Earthquake Engineering Research (PEER) Center framework (Porter 2003, Zhang et al. 2004). A PSDM model is the outcome of probabilistic seismic demand analysis (PSDA) (Mackie and Stojadinovic 2005), and consists in the analytical representation of the relation between a seismic intensity measure (*IM*) and a measure of the structural response of interest, i.e., an engineering demand parameter (*EDP*). In this specific case, the *EPD* of interest is the peak relative displacement between the adjacent buildings. The PSDM can be used to estimate the seismic vulnerability and the mean annual frequency (MAF) of pounding between adjacent buildings via convolution with the hazard curve of the site.

In the development of a PSDM, different choices can be made regarding the *IM* to be employed, the record selection, the technique used in PSDA to estimate the response statistics for different levels of the *IM*, and the model describing the *EDP* statistics given the *IM*. In the present paper, some of these choices are discussed and compared by considering the case of two simple adjacent buildings modeled as single-degree-of-freedom systems with linear or nonlinear hysteretic behavior. Based on the comparison, an optimal demand model is sought as the one that permits to achieve confident estimates of the response statistic with few time-history analyses. This property of the model permits to reduce the complexity and computational cost associated with the evaluation of the risk of pounding between complex nonlinear building models.

2 PROBABILISTIC SEISMIC DEMAND MODEL FOR POUNDING RISK ASSESSMENT

The risk of pounding between two adjacent buildings can be expressed in terms of the MAF, $\nu_{EDP}(\xi)$, with which the relative displacement demand u_{rel} at the most-likely pounding location (*EDP* of interest in this problem) exceeds the separation distance, ξ (Tubaldi et al. 2012). Based on the total probability theorem, $\nu_{EDP}(\xi)$ is expressed as:

$$\nu_{EDP}\left(\xi\right)=\int_{im} G_{EDP|IM}\left(\xi|im\right)\cdot\left|\mathrm{d}\nu_{IM}\left(im\right)\right| \quad (1)$$

in which $G_{EDP|IM}\left(\xi|im\right)$ = complementary cumulative distribution function (CCDF) of $EDP = u_{rel}$ conditional to $IM = im$, and $\nu_{IM}(im)$ = MAF of exceedance of a specific value *im*. In this paper, upper case symbols indicate random variables and lower case symbols denote specific realizations of the corresponding random variable. The probabilistic description of the seismic intensity measure *IM* through the MAF $\nu_{IM}(im)$ is the task of probabilistic seismic hazard analysis and is of major interest for seismologists. The description of $G_{EDP|IM}\left(\xi|im\right)$ is the task of probabilistic seismic demand analysis (PSDA), and returns the PSDM, which is the object of this study.

In general, the computation of $G_{EDP|IM}\left(\xi|im\right)$ involves selecting an *IM* and performing a series of linear or nonlinear time-history dynamic analyses of the structural system of interest under a set of ground-motion records, for different *IM* values. Then, a regression analysis of the *EDP* samples on the corresponding values of *IM* is usually carried out to obtain a synthetic probabilistic description of the seismic demand given *IM* (Luco and Cornell 2007).

The two major issues in defining a PSDM for the problem considered are related to the choice of an appropriate (i.e., efficient and sufficient) *IM*, and to the choice of the regression model for the relation between the *EDP* and the *IM*. It is noteworthy that these two problems are strictly related, because the appropriateness of an *IM*, as described in next section, is usually quantified based on the results of regression analysis, and thus depends on the regression model employed.

3 INTENSITY MEASURES FOR POUNDING RISK ASSESSMENT

The choice of an appropriate *IM* is a critical issue because it affects the computational cost and the reliability of the estimates of $G_{EDP|IM}\left(\xi|im\right)$ and, thus, of $\nu_{EDP}(\xi)$. Usually, the *IM* is selected based on efficiency, sufficiency, and hazard computability criteria (Luco and Cornell 2007, Padgett et al. 2008). The term "efficiency" is related to the dispersion of the seismic demand for a given *IM* value. An efficient *IM* results in a relatively small variability of *EDP* given *IM*, thereby reducing the number of time-history analyses that are necessary to estimate $G_{EDP|IM}\left(\xi|im\right)$ with adequate confidence (Shome and Cornell 1999). The term "sufficiency" refers to the statistical independence of the *EDP* with respect to typical ground motion characteristics such as magnitude (*M*) and source-to-site distance (*R*). For example, if an *IM* sufficient with respect to *M* and *R* is employed for PSDA, Equation (1) can be applied to estimate $G_{EDP|IM}\left(\xi|im\right)$ without having to worry about the values of *M* and *R* of the records employed for non-linear dynamic analyses. The "hazard computability" of an *IM* refers to the availability of a hazard curve or attenuation law for that *IM*, or to the effort required to derive a seismic hazard model in terms of that *IM*. It is noteworthy that the optimal *IM* in terms of efficiency and sufficiency is actually the *EDP* itself (Luco and Cornell 2007). However, directly computing ν_{EDP} via probabilistic seismic hazard analysis would require thousands of time-consuming nonlinear dynamic analyses of the structural model subject to ground motions from an array of *M*'s and *R*'s, which is impractical. Furthermore, this operation should be repeated for each different structure considered. Thus, the best *IM* should be chosen among those for which hazard curves or attenuation laws are readily available or easy to compute. In this paper, a regression model is fitted to the results of PSDA. Thus, the efficiency of the proposed *IM*s is measured by the degree of scatter about the regression fit, whereas their sufficiency is measured by the extent to which the residuals of the regression are statistically independent of *M* and *R* (Luco and Cornell 2007, Vega et al. 2007, Padgett et al. 2008).

Based on the previous considerations, it is advantageous in terms of efficiency to select an *IM* that is as close as possible to the *EDP* of interest. Modal combination rules such as the ABS, SRSS, and DDC rules can provide approximate estimates of the relative displacement response between two adjacent systems in function of their spectral displacement (Lopez-Garcia and Soong 2009a). Since a hazard model is usually available for the spectral displacements, these rules can be employed to define efficient *IM*s for pounding risk assessment.

The simplest *IM* that naturally stems from the use of spectral displacements is:

$$IM_1 = \gamma_A S_d(T_A) \tag{2}$$

where $S_d(T_A)$ denotes the spectral displacement at the fundamental period T_A of the building whose displacement is higher in correspondence of the pounding location, and γ_A denotes the fundamental mode participation factor. In computing γ_A, the modal shape must be normalized to have a unit displacement at the pounding location. This intensity measure is roughly proportional to the spectral acceleration, which is widely employed in PSDA of buildings for its sufficiency and efficiency (Shome and Cornell 1999). However, in the problem considered here, this *IM* could be not appropriate due to the potentially relevant contribution to the peak relative displacement of both buildings.

A potentially more efficient intensity measure can be defined as:

$$IM_2 = \gamma_A S_d(T_A)\sqrt{1+R^2} = IM_1\sqrt{1+R^2} \tag{3}$$

where $R = \left[\gamma_B S_d(T_B)\right] / \left[\gamma_A S_d(T_A)\right]$. This *IM* is very similar to that proposed by Luco and Cornell (2007) to reduce the dispersion of the interstory drift demand in buildings with respect to IM_1, by accounting also for the contribution of the second vibration mode. In this context, IM_2 permits to account for the contribution to the peak relative displacement response of both systems and can be directly related to the SRSS rule for estimating the peak relative displacement.

An even more advanced *IM* can be defined as:

$$IM_3 = IM_1\sqrt{1+R^2-2\rho_{AB}R} \tag{4}$$

where ρ_{AB} denotes the correlation factor between the two system responses (Lopez-Garcia and Soong 2009a). This last *IM* can be directly related to the DDC rule for the peak relative displacement assessment, which is in general more accurate than the ABS and SRSS rules, especially for close fundamental vibration periods (Lopez-Garcia and Soong 2009a, Tubaldi et al. 2012). An attenuation law can be easily derived for the proposed *IM*s if an attenuation law for $S_d(T_i)$ is available. Furthermore, the ratios IM_2/IM_1 and IM_3/IM_1 do not change in the records' scaling, which is controlled by $IM_1 = \gamma_A \cdot S_d(T_A)$ only.

It is noteworthy that the majority of real building structures behaves as multi-degree-of-freedom (MDOF) hysteretic systems. Thus, evaluation of the separation distance between adjacent buildings should account for the contribution of the relevant vibration modes of each building, as well as for their nonlinear behavior. Although several criteria for determining the peak relative displacement between adjacent buildings with nonlinear behavior have been proposed in the literature, none of these criteria appears to be very accurate (Lopez-Garcia and Soong 2009b). In addition, attenuation relations for inelastic spectral displacements are usually not available. Thus, *IM*s based on peak relative displacement estimates that account for nonlinear behavior are not considered in this study. The use of vector valued *IM*s, although of interest for future investigation, is also not considered in this study, because it would open a full range of alternative model forms, combinatorial expansion of the problem considering *IM* pairs, and practical challenges in implementation in a risk assessment analysis procedure, which are all problems beyond the scope of the present study.

4 PSDA AND REGRESSION MODELS FOR POUNDING RISK ASSESSMENT

This study employs the following widely accepted expression for the PSDM that describes the functional relation between *EDP* and *IM* (Cornell et al. 2002):

$$\ln EDP|IM = \ln a + b \ln IM + \ln \varepsilon|IM \tag{5}$$

where the parameters *a* and *b*, as well as the error variable $\varepsilon|IM$ need to be estimated via regression analysis in the log-log space of the *EDP*s samples given *IM*. The functional form given by Equation (5) is based on extensive regression analysis of the seismic response of steel structures (Cornell et al. 2002). The variable $\varepsilon|IM$ is assumed to be lognormally distributed, i.e., $\ln\varepsilon|IM$ follows a normal distribution with zero mean value and standard deviation $\beta_{\ln\varepsilon|IM}(im)$. Thus, also the relative displacement demand follows a lognormal distribution and $\ln EDP|IM$ is normally distributed with mean value $\ln a + b\ln im$ and standard deviation $\beta_{\ln EDP|IM}(im)$.

The assumed regression model permits to evaluate the conditional CCDF, $G_{EDP|IM}(\xi|im)$, of Equation (1) in closed form as:

$$G_{EDP|IM}\left(\xi|im\right)=P\left[EDP|IM\geq\xi|im\right]=$$
$$=\Phi\left(\frac{\ln a+b\cdot\ln im-\ln\xi}{\beta_{\ln EDP|IM}\left(im\right)}\right) \tag{6}$$

where $\Phi(\cdot)$ denotes the standard normal cumulative distribution function.

Different techniques can be used to generate the *EDP*s samples given *IM* (Shome and Cornell 1999, Cornell et al. 2002, Mackie and Stojadinovic 2005). In this study, cloud analysis is employed. The use of this technique is usually coupled with the assumption of homoscedasticity of the demand, i.e., the standard deviation of the *EDP* is assumed constant with respect to *IM* as $\beta_{\ln EDP|IM}\left(im\right)=\beta$ (Mackie and Stojadinovic 2005).

It is noteworthy that, in the case of linear elastic behavior of the two adjacent systems, b can be assumed equal to one, and the PSDM requires a simpler one parameter log-log linear regression. Other studies suggest to take $b = 1$ even for PSDA of systems behaving nonlinearly, e.g., for estimating the maximum interstory drift in building frames for some advanced *IM*s (Luco and Cornell 2007). In this case, the assessment of the efficiency and sufficiency of the *IM* is greatly simplified (Luco and Cornell 2007).

Finally, it is observed that in developing the PSDM, particular attention should be given to the demand samples to be considered. In fact, the buildings are expected to collapse under the action of earthquakes characterized by high *IM*s, i.e., for high building displacement demands. The *EDP*s samples corresponding to these earthquakes should be discarded in the regression analysis.

5 CASE STUDY – ADJACENT BUILDINGS MODELED AS SDOF LINEAR SYSTEMS

In this section, the proposed *IM*s for estimating the relative displacement demand between adjacent buildings modeled as linear elastic single-degree-of-freedom (SDOF) systems are evaluated and compared to each other based on efficiency and sufficiency criteria. An extensive parametric study is performed by considering a wide range of SDOF systems parameters. The choice of employing linear systems is motivated by the fact that the pounding risk becomes high and significant for building which are very close one to each other and, thus, collide for low values of the displacements, corresponding to a linear building response. Furthermore, the use of linear elastic SDOF models for the buildings permit to reduce the number of parameters to be considered in the parametric study.

A dimensional analysis of the problem (Barenblatt 1987) reveals that, using an *IM* whose dimension is a length, the normalized relative displacement response between two buildings undergoing free-vibrations can be expressed as:

$$\frac{u_{rel}}{IM_i}=f\left(\frac{T_{\mathrm{A}}}{T_{\mathrm{B}}},\zeta_{\mathrm{A}},\frac{\zeta_{\mathrm{B}}}{\zeta_{\mathrm{A}}}\right) \qquad i=1,2,3 \tag{7}$$

By contrast, under seismic excitation, the peak relative displacement depends also on the frequency content and duration of the earthquake input, and these effects for a given *IM* are related to both the vibration periods of the two buildings. Thus:

$$\frac{u_{rel}}{IM_i}=f\left(T_{\mathrm{A}},\frac{T_{\mathrm{A}}}{T_{\mathrm{B}}},\zeta_{\mathrm{A}},\frac{\zeta_{\mathrm{B}}}{\zeta_{\mathrm{A}}}\right) \qquad i=1,2,3 \tag{8}$$

To reduce the number of parameters of the analysis, it is assumed here $\zeta_{\mathrm{A}} = \zeta_{\mathrm{B}} = 2\%$. The vibration period of the taller building T_{A} is varied in the range 0-4s, whereas the ratio $T_{\mathrm{B}}/T_{\mathrm{A}}$ in the range 0-0.99999. The value $T_{\mathrm{B}}/T_{\mathrm{A}} = 1$ is not included in the parametric analysis because it corresponds to a discontinuity in the estimate of the dispersion of the response for IM_1 and IM_2. A set of $N_{gm} = 240$ records taken from Baker et al. (2011) is selected to reproduce the variability of the frequency content and duration of the seismic input. Dynamic time-history analyses are carried out under the selected set of records and the results are fitted by a one-parameter linear regression model corresponding to assuming $b = 1$ in Equation (5). The parameter a_i for the i-th *IM* (i = 1, 2, 3) is estimated as the 50th percentile of the samples of the normalized demand u_{rel}/IM_i, whereas the lognormal standard deviation or dispersion β_i is evaluated as (Luco and Cornell 2007):

$$\beta_i=\sqrt{\frac{\sum_{j=1}^{N_{gm}}\left(\ln\left(u_{rel}/IM_i\right)_j-\ln\left(a_i\right)\right)^2}{N_{gm}-2}} \qquad i=1,2,3 \tag{9}$$

Figure 1 reports the normalized median response a_i vs. T_{A} and $T_{\mathrm{B}}/T_{\mathrm{A}}$, for the different *IM*s considered. Figure 1(a) shows the results obtained by considering the peak ground acceleration (*PGA*) of the records as *IM*. Since the dimension of *PGA* are of a length divided by a squared time, the relative displacement is normalized as $u_{rel}\omega_{\mathrm{A}}^2/im$.

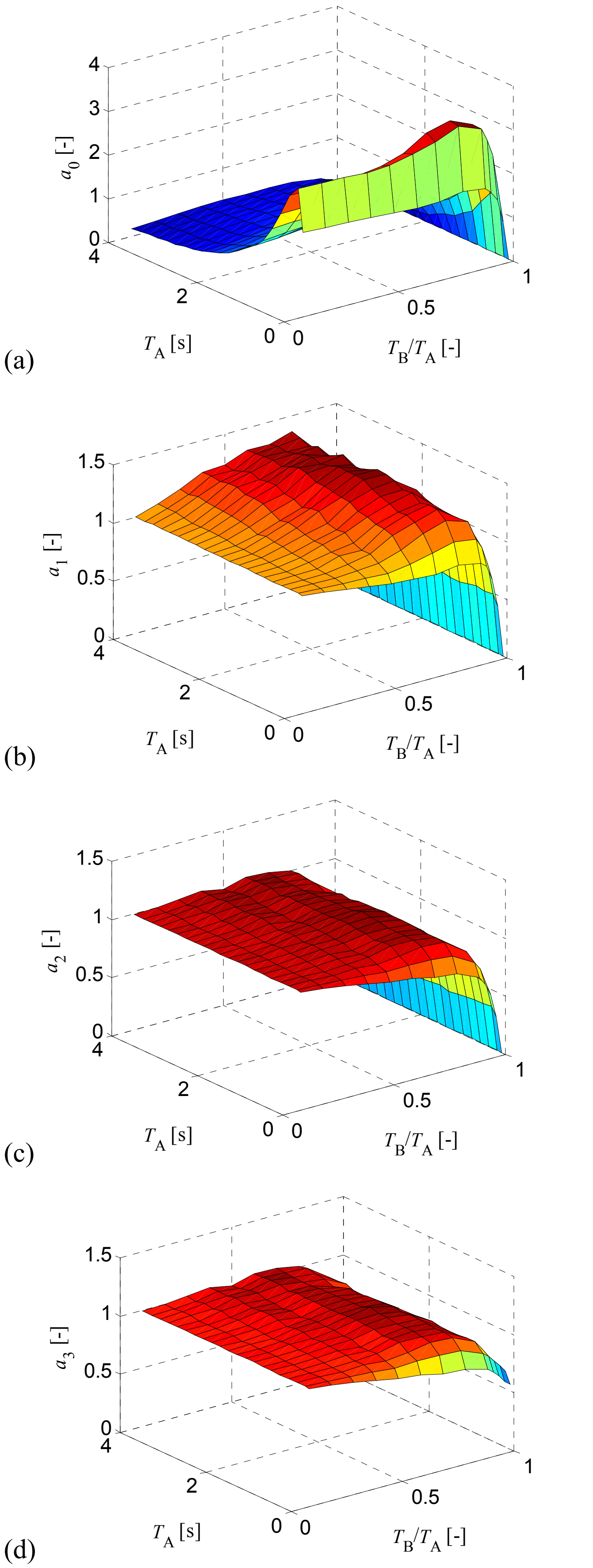

Figure 1. Normalized median relative displacement for different system vibration periods obtained by assuming as *IM*: (a) *PGA*, (b) IM_1, (c) IM_2, (d) IM_3.

It is observed that, for the *IM*s based on spectral displacements (i.e. IM_1, IM_2, IM_3), the values of the normalized displacement demand a_i (i = 1, 2, 3) are almost insensitive to the vibration period T_A of the taller building. They slowly increase when T_B/T_A increases from 0 to approximately 0.8 and decrease when T_B/T_A increases from 0.8 to 1. For $T_B / T_A = 0 \div 0.8$ and $T_A \geq 0.3\text{s}$, IM_2 and IM_3 are only slightly biased in estimating a_i (i.e., a_i assumes values close to one for i = 2, 3), whereas IM_1 is more biased, because it does not take into account the contribution of system B to the relative displacement response. In the same period ranges, IM_2 practically coincides with IM_3, because the correlation factor ρ_{AB} is almost zero for distant vibration periods. As T_B/T_A approaches zero, the normalized peak displacements a_i (i = 1, 2, 3) tend to slightly less than one. This is due to the fact that the relative displacement tends to the displacement of building A, while IM_i (i = 1, 2, 3) approach the peak absolute displacement of building A. Thus,

$$\lim_{T_B/T_A \to 0} a_i = \lim_{T_B/T_A \to 0} \mathrm{E}\left[\frac{\max_t\left(u_A\right)}{\max_t\left|u_A\right|}\right] \leq 1; \quad i = 1, 2, 3 \tag{10}$$

For T_B/T_A approaching one from below, the normalized relative displacement response tends to zero if IM_1 or IM_2 are employed, because the two systems vibrate in phase. IM_3 is less biased in estimating the peak displacement, because it accounts for the correlation between the adjacent buildings' responses. For T_B/T_A approaching one, IM_3 tends to zero. However, a_3 tends to a finite value which depends on the system and ground motion properties (in fact, the DDC rule and thus IM_3 provide exact estimates of the peak relative displacement only in the case of stationary response to stationary white noise excitation). The displacement demand normalized to the *PGA* exhibits a significant dependence on both T_A and T_B/T_A. It is noted that for T_B/T_A = 0, the values of a_0 vs. T_A in Figure 1(a) coincide with the median pseudo spectral accelerations of the records.

Figure 2 reports the dispersions β_i vs. T_A and the ratio T_B/T_A, for the different *IM*s considered. In general, the dispersion β_0 (for *IM* = *PGA*) is very high (Figure 2(a)). On the other hand, β_i is quite low for *IM*s based on spectral displacements (Figure 2(b-d)). For T_B/T_A in the range between 0 and 0.8, β_1 assumes values below 0.30, and β_2 and β_3 assume values below 0.2. The higher efficiency of IM_2 and IM_3 is due to the fact that they account for the contribution of the shorter building to the relative displacement demand. For T_B/T_A approaching one from below, β_i increases significantly for i = 1, 2, 3, and IM_3 does not appear significantly more efficient than IM_2.

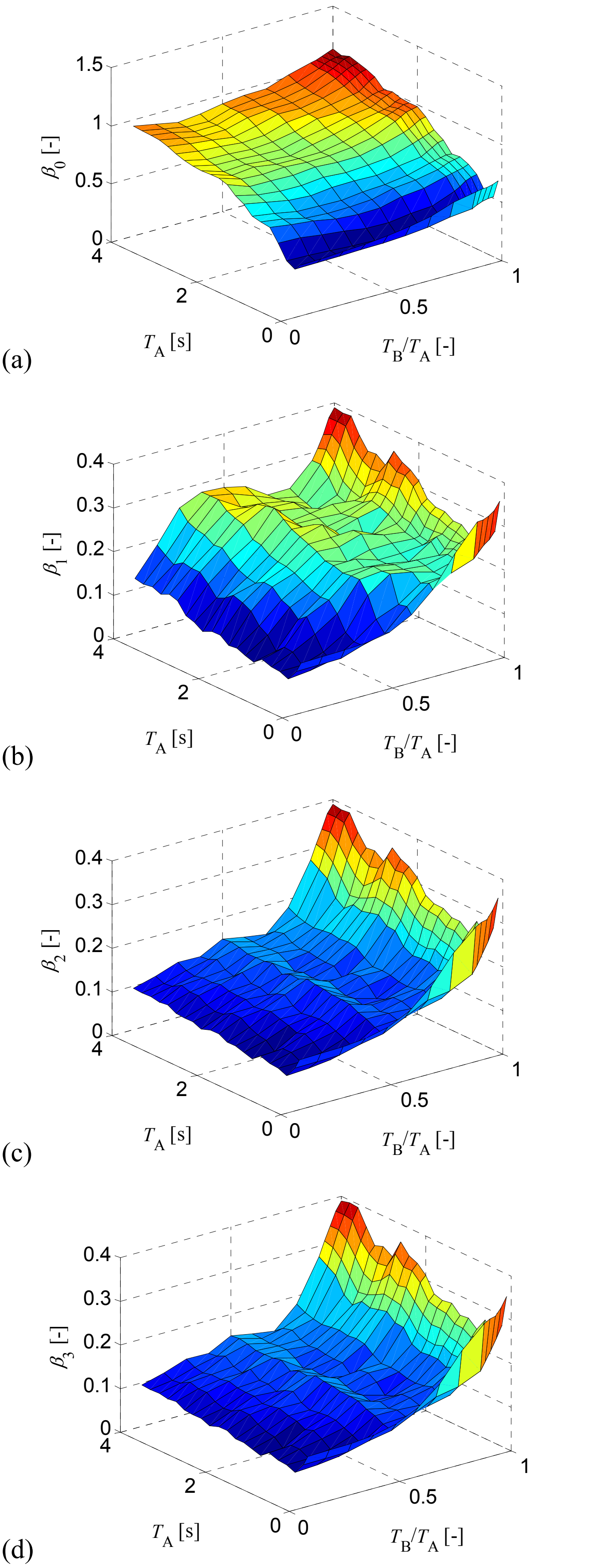

Figure 2. Values of the relative displacement response dispersion β for different system vibration periods obtained by assuming as *IM*: (a) *PGA*, (b) IM_1, (c) IM_2, (d) IM_3.

The study of sufficiency of the *IM*s with respect to *M* and *R* has been carried out by following the procedure reported in Luco and Cornell (2007). The results of this study, not reported here due to space constraint, show that IM_2 and IM_3 are superior compared to IM_1 also in terms of sufficiency for a wide range of system properties. IM_0 is largely insufficient with respect to both *M* and *R*, irrespectively of the system properties.

6 CASE STUDY – ADJACENT BUILDINGS MODELED AS SDOF HYSTERETIC SYSTEMS

In this section, PSDA is applied to evaluate the PSDM for the case study of two buildings with nonlinear hysteretic behavior. The same buildings already analyzed in Barbato and Tubaldi (2012) are considered here. Building A is an eight-story shear-type building with constant inter-story stiffness k_A = 628,801kN/m and floor mass m_A = 454.55kNs^2/m, while building B is a four-story shear-type building with constant inter-story stiffness 470,840kN/m and floor mass m_B = 454.55kNs^2/m. A Rayleigh-type damping matrix $\mathbf{c_R}$ is used to model the inherent viscous damping in the two systems. The matrix is built by assigning a damping factor ζ_R = 2% to the first two vibration modes of each system considered independently from the other. The fundamental vibration periods of the two buildings are T_A = 0.915s and T_B = 0.562s, respectively.

In order to minimize the computational cost of the procedure, reduced-order SDOF models are developed for the two buildings based on their fundamental vibration mode, as in Barbato and Tubaldi (2012). The relative displacement of the buildings corresponding to their first vibration modes only are:

$$U_{rel}(t) = \gamma_A \cdot V_A \cdot Z_A(t) - \gamma_B \cdot V_B \cdot Z_B(t) \qquad (11)$$

where γ_A, γ_B = participation factors of the first modes of building A and B, respectively; V_A, V_B = first mode shape displacements at the pounding location (normalized by the first mode displacement at the roof) of building A and B, respectively; and $Z_A(t)$, and $Z_B(t)$ = time-histories of the generalized coordinate corresponding to the roof displacement according to the first mode of vibration of the two buildings and the specified seismic input.

The two equivalent SDOF systems corresponding to buildings A and B are characterized by the following properties: $T_{eq,A}$ = 0.915s and $T_{eq,B}$ = 0.562s (fundamental vibration periods), $m_{eq,A}$ = 3,113.9kNs^2/m and $m_{eq,A}$ = 1,624.4kNs^2/m (equivalent masses), $\zeta_{eq,A} = \zeta_{eq,B}$ = 2% (equivalent damping ratio), and $\gamma_A \cdot V_A$ = 0.853, $\gamma_B \cdot V_B$ = 1.241. A bilinear constitutive model with kinematic hardening describes the relationship between the inelastic restoring force and the equivalent SDOF system displacements (Lopez-Garcia and Soong 2009b). This constitutive model for building i (with i = A, B) is defined by the yield displacement, $F_{y,i}$, and by the ratio of the post-yield and initial stiffnesses, b_i, which is assumed equal to 0.05 for both models. The yield forces for system A and B are respectively $F_{y,\mathrm{A}}$ = 6871.4kN and $F_{y,\mathrm{B}}$ = 2716.5kN and are derived from Lin (2005).

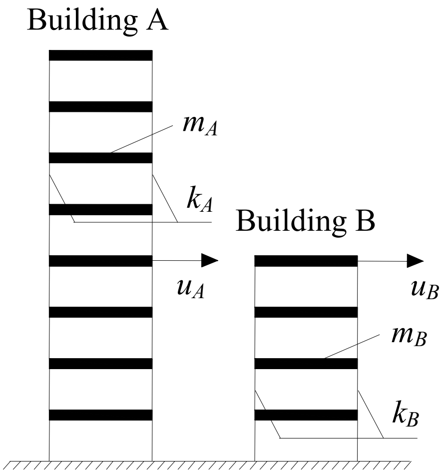

Figure 3. Models of buildings A and B.

Cloud analysis is applied to this case study by employing the same set of records already considered in the previous section. The ratio R_i between the yield displacement of system i and the median spectral displacement of the records are respectively equal to 0.9304 and 1.4790 for system A and B. Thus, the buildings are expected to undergo significant inelastic deformations for a high number of records. The *EDP* samples corresponding to values of the displacement ductility demand for the systems higher than 5 are discarded in developing the PSDM.

Figure 4 shows the two-parameters linear regression models obtained by considering all the *EDP* samples (unconditioned PSDM) and the reduced set of samples conditioned on not exceeding the ductility capacity of 5 by the two systems (conditioned PSDM).

It is observed that the unconditioned and conditioned PSDMs are very similar. This can be explained by noting that the ductility capacity of 5 is exceeded only for 14 of the 240 records considered in cloud analysis. By disregarding these 14 cases, the dispersion of the demand is reduced, due to the reduced excursion into the nonlinear range of the response of the two systems. It is also noted that the values of the regression parameter b are inferior to one and span from 0.6 to 0.7.

(a)

(b)

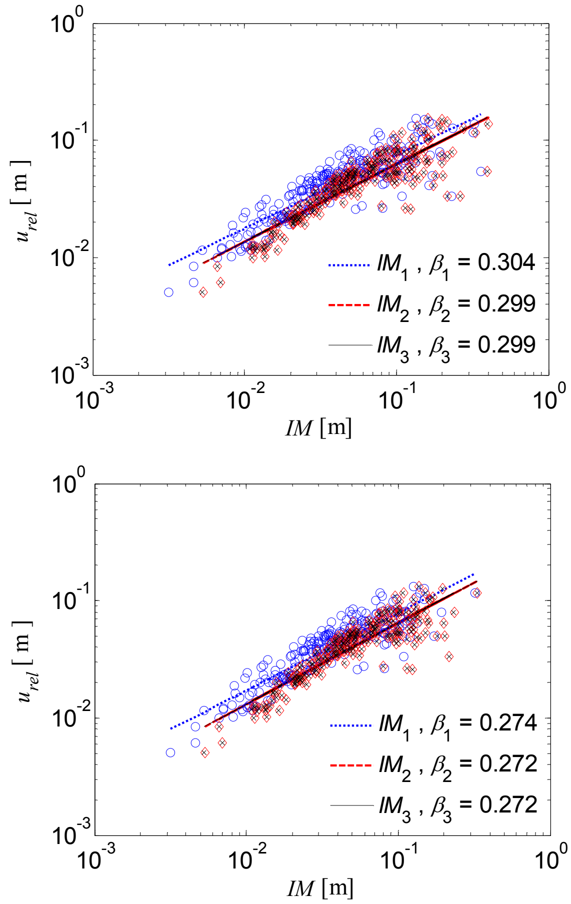

Figure 4. PSDM of the peak relative displacement demand obtained by using IM_1, IM_2, and IM_3 and by considering (a) all the *EDP* samples, (b) the *EDP* samples conditional to not exceeding the system ductility capacity.

In order to evaluate the sufficiency of the *IM*s with respect to M and R, a linear regression of the residuals $\ln \varepsilon_j \big| IM_i$, for $j = 1, 2, .., N_{gm}$, is computed separately on M and R (Vega et al. 2009). The *IM* sufficiency can be quantified by the p-value, which is defined as the probability of finding a regression slope term as large as the one already found when its real value is zero, which is the null hypothesis. A small p-value suggests that the estimated slope term is statistically significant, and that the IM_i is insufficient. Figure 5 and Figure 6 show the residuals together with the regression lines and p-values evaluated respectively for M and for R.

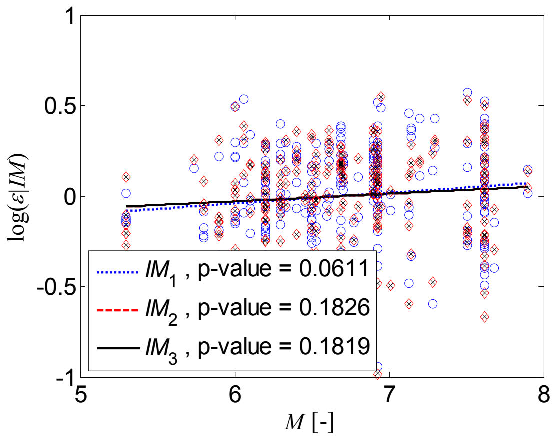

Figure 5. Regression of residuals on magnitude M.

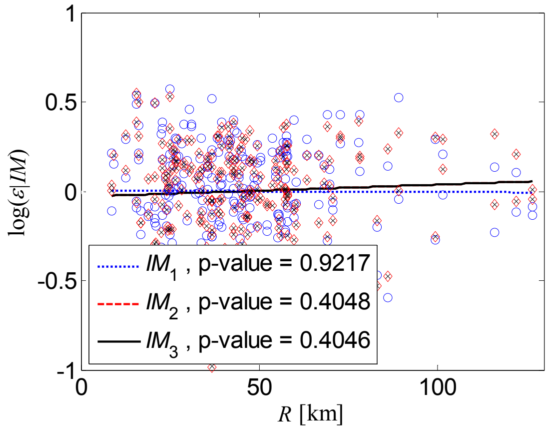

Figure 6. Regression of residuals on source-to-site distance *R*.

Based on a value of 0.05 as cut-off for the p-values, the null hypotheses that the slopes of the regression lines are zero cannot be rejected for any of the three *IM*s considered. Thus, for this example, these *IM*s are sufficient with respect to both *M* and *R*. In particular, IM_2 is the most sufficient with respect to *M*, whereas IM_1 is the most sufficient with respect to *R*.

7 CONCLUSION

This paper aims at providing some insight into the choice of a suitable intensity measure (*IM*) and probabilistic demand model for assessing the risk of pounding between adjacent buildings within the PEER PBEE framework. Different *IM*s are proposed based on widespread code rules for estimating the building's separation distance. The optimal *IM* selection is carried out by assuming a demand model often employed in probabilistic seismic demand analysis to describe the relation between the system response and the *IM*.

A parametric study considering the seismic response of adjacent buildings modeled as SDOF linear systems under a suite of 240 natural ground motion records reveals that IM_2 and IM_3, based respectively on the SRSS and DDC code rules, are superior in terms of efficiency to more common *IM*s (i.e., the peak ground acceleration, $IM_0 = PGA$, and the spectral displacement at the fundamental period of the taller building, IM_1). Additional analyses carried out on equivalent nonlinear SDOF models of two realistic buildings confirm that IM_2 and IM_3 are superior to IM_1 in terms of efficiency and that IM_1, IM_2, and IM_3 are sufficient with respect to the magnitude and the source-to-site distance.

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