16th World Conference on Earthquake Engineering, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 2233

Registration Code: S-X1464106625

TOWARDS A PRACTICE-ORIENTED STRATEGY TO INCLUDE LOSSES IN THE SEISMIC ASSESSMENT OF RC FRAME BUILDINGS

N. Pereira [(1)], X. Romão [(2)]

[(1)] PhD Student, Faculty of Engineering of the University of Porto, Portugal, nmsp@fe.up.pt

[(2)] Assistant Professor, Faculty of Engineering of the University of Porto, Portugal, xnr@fe.up.pt

Abstract

One of the main objectives of Performance-Based Earthquake Engineering (PBEE) is to allow for the direct analysis of a set of decision variables characterizing the response of a structure located in a specific site. Among these decision variables, loss metrics are particularly important due to their direct connection to the decisions related with retrofitting needs and post-earthquake reparability issues. Despite the recognized importance of these decision variables (DV) for stakeholder information and to develop adequate mitigation strategies, their inclusion in current seismic assessment frameworks for RC frame buildings is still limited when assessing the performance of both new and older structures. Conversely, most of the assessments are based on engineering demand parameters (EDPs) such as interstorey drifts or component deformations, which are used as benchmarks to classify the damage states and the overall performance of a building.

However, the information provided by EDPs is insufficient to support adequate decision-making by stakeholders, mostly because they do not provide objective estimates for the potential post-earthquake reparability needs and for the respective losses/costs they may involve. In some cases, the importance of losses and repair costs might even be larger than that of the probability of collapse of the building when exposed to a given scenario. As an example, for lower seismic hazard levels, a structure may present inadequate damage accumulation which may lead to excessive repair needs or even to the decision of replacing it by a new structure, while exhibiting a low probability of collapse.

With this in mind, the present paper analyses how decision variables can be included in the assessment of the expected losses or of loss-based limit states in RC frame buildings. Specifically, the derivation of simplified analytical tools is addressed, since they provide expressions that are simple to apply and allow for a widespread use of PBEE methods within the technical community. Moreover, such analytical expressions do not require extensive knowledge on probabilistic analysis or the use of numerical integration procedures. Methodologies to include loss estimates into standard intensity-based assessments and in limit state compliance criteria for the evaluation of the mean annual frequency of exceedance are developed in the paper. A first approximation is analysed using a storey-based loss assessment approach to verify how to analytically estimate the losses based on the demand results obtained from an intensity-based assessment. An analytical function is fitted to EDP-DV datasets available in the literature and an expression is proposed to compute the expected value of the losses using the distribution of the interstorey drifts evaluated at a stripe of ground motions (intensity-based assessment). Secondly, a new set of limit state conditions is analysed using the referred analytical storey-based loss assessment approach combined with incremental dynamic analysis and scalar loss variables.

Keywords: Reinforced Concrete, frame buildings, Performance-based Earthquake Engineering, repair costs, losses.

1. Introduction

Performance-based seismic assessment of buildings is a methodology that targets the evaluation of the seismic performance of a system when it is subjected to earthquake ground motions with different intensities. The main reason for the development of these methods stemmed from the observations made in the aftermath of the Loma Prieta 1989 and the Northridge 1994 earthquakes in California. After these events, stakeholders started to express some concerns regarding the performance of the buildings, namely the damage they presented among structural and non-structural components, despite ensuring life-safety conditions. Within this context, the main principles of performance-based earthquake engineering (PBEE) were developed, including explicit or implicit references to the human and economic consequences of the ground motion effects. One of the results of this new philosophy was the definition of performance matrices as a complement to traditional safety assessment methods. These matrices, which were first introduced in American standards [1, 2], define the maximum damage that is allowed to occur in the structure as a consequence of a ground motion with a given intensity.

In Europe, one of the main developments introduced in order to create a unified approach to assess the seismic safety of existing RC buildings was the publication of Part 3 of Eurocode 8 (EC8/3) [3]. Similarly to other standards available worldwide, the EC8/3 establishes 3 classes of performance objectives classified in a decreasing order of observed damage as Near Collapse (NC), Significant Damage (SD) and Damage Limitation (DL). These performance levels are qualitatively described in the standard in terms of admissible damage levels and deformations, and each class of performance objectives is connected to a specific level of seismic hazard (represented by a specific average return period). For NC, the performance objectives require the structure to still be able to sustain gravity loads after the ground motion, even though it may exhibit heavy damage and large permanent deformations. Conversely, the performance objectives associated with the class of SD refer that non-structural components are expected to exhibit significant damage (although without out-of-the-plane collapse of infill walls). A structure compatible with the SD class is also expected to exhibit residual interstorey drifts with a moderate magnitude while still being able to sustain a moderate intensity aftershock without collapsing. EC8/3 also states that a structure exceeding the limit conditions associated with SD has a significant probability of being uneconomic to repair. Finally, the performance objectives associated with DL establish that no damage is expected in the structural elements while only minor damage such as cracking of the infill walls is expected for the non-structural components. Consequently, DL implies a post-earthquake state of the building with very low repair needs and assumes that no residual deformation has occurred in the building.

The transitions between the performance or damage levels defined in EC8/3 are characterized by limit state conditions that establish limits above which the building is no longer compatible with a given performance class. Table 1 presents the referred damage states and the limits of corresponding compliance criteria.

Table 1 – Damage states and performance objectives defined in the current version of EC8/3.

Performance Objectives	**Structural Components**	**Non-structural Components**	**Permanent deformations**	**Reparable structure?**	**Compliance criteria**
Damage Limitation, DL	Light	Economical repair	Negligible	Yes	θ_{DL}
Significant Damage, SD	Significant	Damaged	Visible	Uneconomic	θ_{SD}
Near Collapse, NC	Heavy	Collapsed	Large	No	θ_{NC}, V_{NC}

As shown in Table 1, the structure can be said to comply with the performance objectives of DL if the corresponding limit state value θ_{DL} is not reached at any element. For ductile elements, it can be seen that the compliance criteria defining the transition between the several damage limit states are defined in terms of local deformations (chord rotations, θ). However, for the case of NC, the fragile failure of structural elements must also be analysed by assessing the shear demand of the elements. These verifications are defined only for structural elements and connected to a specific level of seismic hazard compatible with the limit states establishing the separation between the performance classes. If a single component exceeds a given limit state condition, the overall building is classified as non-compliant with that limit state. Hence, the considerations made regarding the state of non-structural components, the level of residual deformations, the capacity reserve against collapse or the level of repair losses are not explicitly included in the limit state verifications.

In light of this, the present paper aims to analyse a possible strategy to create a practice-oriented methodology to explicitly include information about losses in the seismic safety assessment of RC frame buildings. Specifically, the proposed methodology derives a set of limit state conditions and the corresponding compliance criteria that allow for the explicit verification of the EC8/3 performance objectives and include all the variables that are qualitatively described in their definition. Moreover, the methodology is not only able to be used in an intensity-based assessment procedure as the one currently proposed in EC8-3 but it can also directly include risk-based metrics in the LS verification as an alternative.

2. Evaluation of seismic repair costs

Despite being associated with multiple definitions [4], performance-based methods often refer to the so-called PEER-PBEE methodology. The Pacific Earthquake Engineering Research Centre (PEER) methodology was developed to respond to the need for communicating seismic risk to stakeholders involving metrics that reflect the seismic consequences at a scale different than the engineering terms that are usually adopted in earthquake engineering. This methodology allows for the quantification, in probabilistic terms, of different decision variables (DVs) such as monetary losses, repair time or number of fatalities. The basis of the PEER methodology lies in the probabilistic characterization of several performance metrics along with the multiple sources of uncertainty that are inherent to any seismic evaluation (e.g. the uncertainty about the hazard, about the ground motions representing a seismic scenario, modelling and knowledge-based uncertainties about the building components and properties) [5]. The PEER methodology can be resumed into the framing equation representing the rate of a certain DV exceeding a given value *dv* [6] defined by:

$$\lambda\left(\mathrm{DV}>\mathrm{dv}\right)=\int_{\mathrm{IM}}\int_{\mathrm{EDP}}\int_{\mathrm{DM}}\mathrm{G}\left(\mathrm{DV}|\mathrm{DM}\right)\cdot\left|\mathrm{dG}\left(\mathrm{DM}|\mathrm{EDP}\right)\right|\cdot\left|\mathrm{dG}\left(\mathrm{EDP}|\mathrm{IM}\right)\right|\cdot\left|\mathrm{d}\lambda\left(\mathrm{IM}\right)\right| \tag{1}$$

where DM represents a damage measure, generally discretised into several damage states, EDP represents a measure of the structural response which can be correlated with DM, IM is a ground motion intensity measure and G(·) is the complementary cumulative distribution function. The numerical integration of Eq. (1) can be used to estimate the annual losses. By discretizing the IM domain, the expected annual losses (EAL) can be calculated as:

$$EAL=\sum_{i=1}^{n}E\left(L\,|\,IM_i\right)\cdot p\left(IM_i\right) \tag{2}$$

where $p(IM_i)$ includes information about the seismic hazard level and $E(L|IM_i)$ is the expected value of the losses for an event with an intensity IM_i. Solving Eq. (2) requires quantifying the expected value of the losses, $E(L|IM_i)$, for each ground motion intensity IM_i, which can be estimated from the proposal of Ramirez and Miranda [7], based on the previous work by Aslani [8]:

$$\begin{aligned}E\left(L\,|\,IM_i\right)=&\,E\left(L\,|\,\bar{C}\cap R,IM_i\right)\cdot p\left(\bar{C}\cap R\,|\,IM_i\right)+\\&+E\left(L\,|\,\bar{C}\cap D,IM_i\right)\cdot p\left(\bar{C}\cap D\,|\,IM_i\right)+\\&+E\left(L\,|\,C,IM_i\right)\cdot p\left(C\,|\,IM_i\right)\end{aligned} \tag{3}$$

with $E\left(L|\bar{C}\cap R,IM_i\right)$, $E\left(L|\bar{C}\cap D,IM_i\right)$ and $E(L|C,IM_i)$ are the expected value of the losses for IM_i given that the structure is still reparable (without collapsing), the expected value of the losses for IM_i given that the structure is not reparable (without collapsing) and the expected value of the losses for IM_i given that the structure will collapse, respectively. The probabilities of having a reparable and an irreparable building without collapsing can be calculated by factorizing the corresponding probability of demolition, $p\left(D \mid \bar{C}, IM_i\right)$, by the probability of collapse, $p\left(C \mid IM_i\right)$, as show in Eqs. (4) and (5).

$$p\left(\bar{C} \cap R \mid IM_i\right) = \left[1 - p\left(D \mid \bar{C}, IM_i\right)\right] \cdot \left[1 - p\left(C \mid IM_i\right)\right] \tag{4}$$

$$p\left(\bar{C} \cap D \mid IM_i\right) = p\left(D \mid \bar{C}, IM_i\right) \cdot \left[1 - p\left(C \mid IM_i\right)\right] \tag{5}$$

Finally, the expected value of the losses for a given ground motion intensity IM_i can be quantified considering a relative quantity, the loss ratio, which consists of the ratio between the obtained losses and the cost of replacing the structure. This implies that the loss ratio is 1.0 for the case where the structure is considered irreparable and for the case where a structural collapse is observed, with Eq. (3) becoming:

$$\begin{aligned} E\left(L \mid IM_i\right) = & \, E\left(L \mid \bar{C} \cap R, IM_i\right) \cdot \left[1 - p\left(D \mid \bar{C}, IM_i\right)\right] \cdot \left[1 - p\left(C \mid IM_i\right)\right] + \\ & + p\left(D \mid \bar{C}, IM_i\right) \cdot \left[1 - p\left(C \mid IM_i\right)\right] + \\ & + p\left(C \mid IM_i\right) \end{aligned} \tag{6}$$

3. Proposed strategy to include losses into the EC8/3 framework

A closer analysis of the EC8/3 performance objectives (Table 1) shows that most variables currently used to estimate losses, as those included in Eq.(6), have to be controlled for each limit state. Particularly, as seen in Table 1, references are made to repair costs, excessive residual deformations and reparability conditions. Furthermore, a clear indication about the damage levels that are admissible for each type of component (structural and non-structural) is provided. Nonetheless, as denoted in [9-11], the compliance criteria involving a single member failure condition in terms of chord rotations and shear strength values do not consistently represent the intended damage conditions. Hence, a review of the code procedure requires adopting compliance criteria that involve, in a consistent manner, the performance requirements that are inherent to the code safety assessment procedure.

Currently, the EC8/3 compliance criteria can be seen as a set of scalar damage variables [12] which correlate the element demand and the corresponding limit state capacity:

$$Y_{LS} = \frac{\Theta \mid \mathrm{IM}_{LS}}{\Theta_{LS}} \tag{7}$$

These scalar damage variables must be calculated for every component of the building and will deem the building as non-compliant if the scalar variable Y_{LS} of a single structural component exceeds 1.0. A similar rationale can be adopted to define a new Y_{LS} condition involving a direct loss-based approach instead of an EDP(Θ)-based one. Accordingly, Y_{LS} can be defined in terms of expected loss for a given intensity using a limit value of average admissible loss $L_{m,LS}$ representing the corresponding LS capacity:

$$Y_{LS} = \frac{E\left(L \mid \mathrm{IM}_{LS}\right)}{L_{m,LS}} \tag{8}$$

Limit values for the average admissible loss $L_{m,LS}$ can be defined in order to comply with some of the EC8/3 performance objectives as shown in Table 2. For the DL performance objective, a limit value of $L_{m,DL}$=0.10 is proposed in order to comply with the damage limitation on non-structural components. Since these components have to remain reparable, a limited amount of damage is admissible in these components. Since many non-structural systems (piping, electricity, windows and doors) depend on the performance of infill walls in RC frame buildings, $L_{m,DL}$ will be associated with a limited value of the maximum interstorey drift (IDR). This condition will also limit the damage in structural components to minor cracking, since a global loss of 10% will be mostly disaggregated among the repair of non-structural components. Also, limited values of peak floor acceleration (PFA) must also be observed to limit the non-structural damage of electric components and ceilings.

For SD, EC8/3 establishes a major condition regarding the reparability of the building. Accordingly, a structure not complying with the SD limit state condition is considered to be uneconomic to repair. Hence, an economics-based limit state is clearly established by EC8/3 and it means there is a high probability of demolishing the structure when the SD limit state condition is achieved. Therefore, a direct loss-based limit value can be established following, for example, the information provided in the FEMA P58 [13]. According to [13], based on the studies of Ramirez and Miranda [9], when a building presents a loss ratio of 0.40-0.50, stakeholders are keener to replace the building than to restore it to an undamaged configuration. Thus, $L_{m,SD}$=0.40 can be defined to represent the SD performance objective.

According to the EC8/3 definition, there is no specific reason to include a loss-based condition to verify the NC performance objective. For this performance objective, all elements will present heavy damage. Therefore, controlling the elements' damage can be seen as an adequate proxy to evaluate the NC limit state. Thus, all the requirements for the NC performance objective can be translated into the strength or deformation-based conditions currently established in EC8/3. Table 2 summarizes the modified compliance criteria adopted hereon, along with the corresponding damage states considered by the EC8/3 performance objectives.

Table 2 – Modified damage states and performance objectives compatible with EC8/3.

Performance Objectives	**Structural Components**	**Non-structural Components**	**Permanent deformations**	**Reparable structure?**	**Compliance criteria**
Damage Limitation, DL	Light	Economic repair	Negligible	Yes	$L_{m,DL}$=0.10
Significant Damage, SD	Significant	Damaged	Visible	Uneconomic	$L_{m,DL}$=0.50
Near Collapse, NC	Heavy	Collapsed	Large	No	θ_{NC}, V_{NC}

3.1. Calculating the seismic demands using a loss condition

To complete the verification of the Y_{LS} condition, an efficient strategy needs to be developed to quantify the demand parameters from the analysis results. Considering the results of Eq.(6), the quantification of $E(L|IM_{LS})$ has to integrate information about the probability of demolition, about the probability of collapse and about the repair losses. In the present paper, focus is put on the seismic response quantification using nonlinear dynamic analysis. In this particular case, if N_{gm} ground motions are adopted, the scalar loss variable Y_{LS} can be re-written as:

$$Y_{LS} = \frac{1}{N_{gm}} \cdot \sum_{n_{gm}=1}^{N_{gm}} Y_{LS,gm} = \frac{1}{N_{gm}} \cdot \sum_{n_{gm}=1}^{N_{gm}} \frac{L \mid n_{gm}}{L_{m,LS}} \tag{9}$$

where $L|n_{gm}$ represents the losses associated with the response of the structure to ground motion n_{gm} with intensity IM_{LS}. By adding all the losses estimated for each ground motion and dividing the sum by the number of considered accelerograms N_{gm} (which, ideally, must be around 20-30), the corresponding expected value of the losses for a ground motion with intensity IM_{LS} can be estimated.

For each ground motion, the quantification of $L|n_{gm}$ must include all the variables previously analysed. In order to comply with this condition, the following expression is proposed:

$$L \mid n_{gm} = \begin{cases} E\left[\mathrm{L} \mid n_{gm}\right] \cdot \left[1 - \Phi\left(\dfrac{\ln\left[RIDR\right] - \ln\left[0.015\right]}{0.3}\right)\right] + \Phi\left(\dfrac{\ln\left[RIDR\right] - \ln\left[0.015\right]}{0.3}\right) & \text{if } \overline{C} \\ 1.0 & \text{if } C \end{cases} \tag{10}$$

which includes the expected value of the repair costs $E[L|n_{gm}]$, the loss component related to the probability of demolition given the permanent deformations (*RIDR*) exhibited by the building when subjected to ground motion n_{gm} and, indirectly, the probability of collapse by assigning a value of 1.0 to $L|n_{gm}$ if one of the NC criteria is reached. In the proposed methodology, the probability of demolition was represented according to the Ramirez and Miranda [9] proposal. Hence, the probability of demolition can be calculated using the value of the maximum residual IDR (RIDR) considering that the aforementioned probability follows a lognormal distribution with mean RIDR= 0.015 and a dispersion of 0.30. Figure 1 summarizes the main steps necessary to verify the DL and SD limit state conditions.

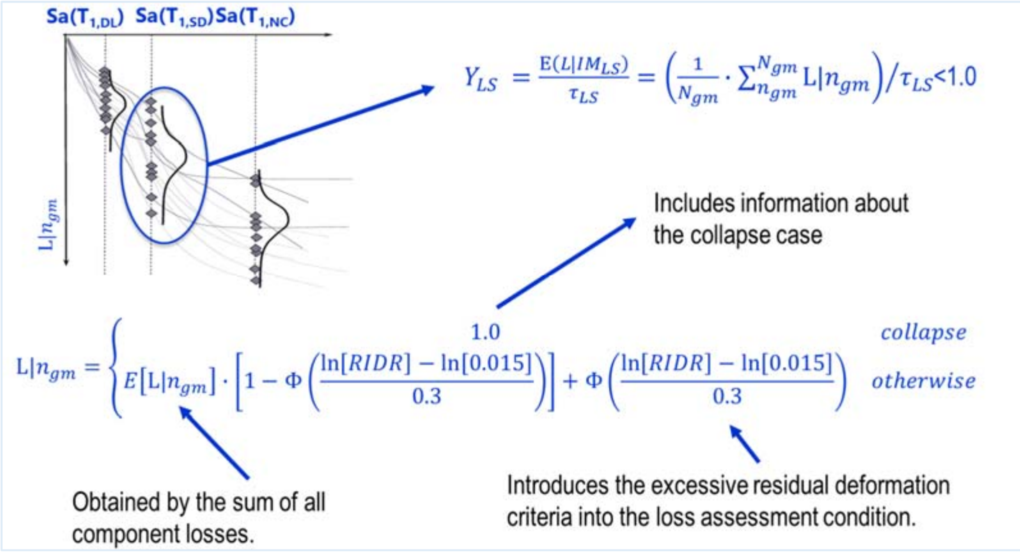

Figure 1. Main procedure to estimate Y_{LS} for the

As shown in Fig. 1, the limit state condition is only completely defined with the calculation of the expected value of the repair costs, $E[L|n_{gm}]$, which can be obtained from the sum of the repair costs of each individual structural and non-structural component. Hence, its quantification requires making an inventory of all the components of the building and the derivation of fragility functions and consequence models for every type of component. Since an extensive inventory of the building components can be difficult to obtain, alternative methods, such as those based on the storey losses proposed by Zareian and Krawinkler [14] and Ramirez and Miranda [15] can be used. The main principle behind storey-based loss assessment methods consists in assuming that the total losses can be computed from the sum of the repair costs at each storey and floor of the building. Furthermore, it assumes that components can be grouped into 3 categories: IDR-sensitive structural (S|IDR) and non-structural components (NS|IDR) and PFA-sensitive non-structural components (NS|PFA). These categories are then weighted in order to reflect the amount of each component category that can be found in a given storey. By combining several components, Ramirez and Miranda [15] developed several sets of data for the 3 categories for multiple types of RC frame office buildings. One important observation that can be made about these datasets addresses the shape of the IDR and PFA EDP-to-Loss functions. Particularly, in order to derive a practice-oriented method, an analytical strategy needs to be established to compute the losses associated with IDR and PFA. Hereon, the following analytical model is proposed to calculate the above-mentioned losses based on IDR and PFA demands:

$$L \mid EDP = k_0 \cdot \exp\left[-k_2 \cdot \ln^2 EDP - k_1 \cdot \ln EDP\right] \tag{11}$$

The adequacy of the proposed model when fitted to the data developed by Ramirez and Miranda [15] can be evaluated from Fig. 2.

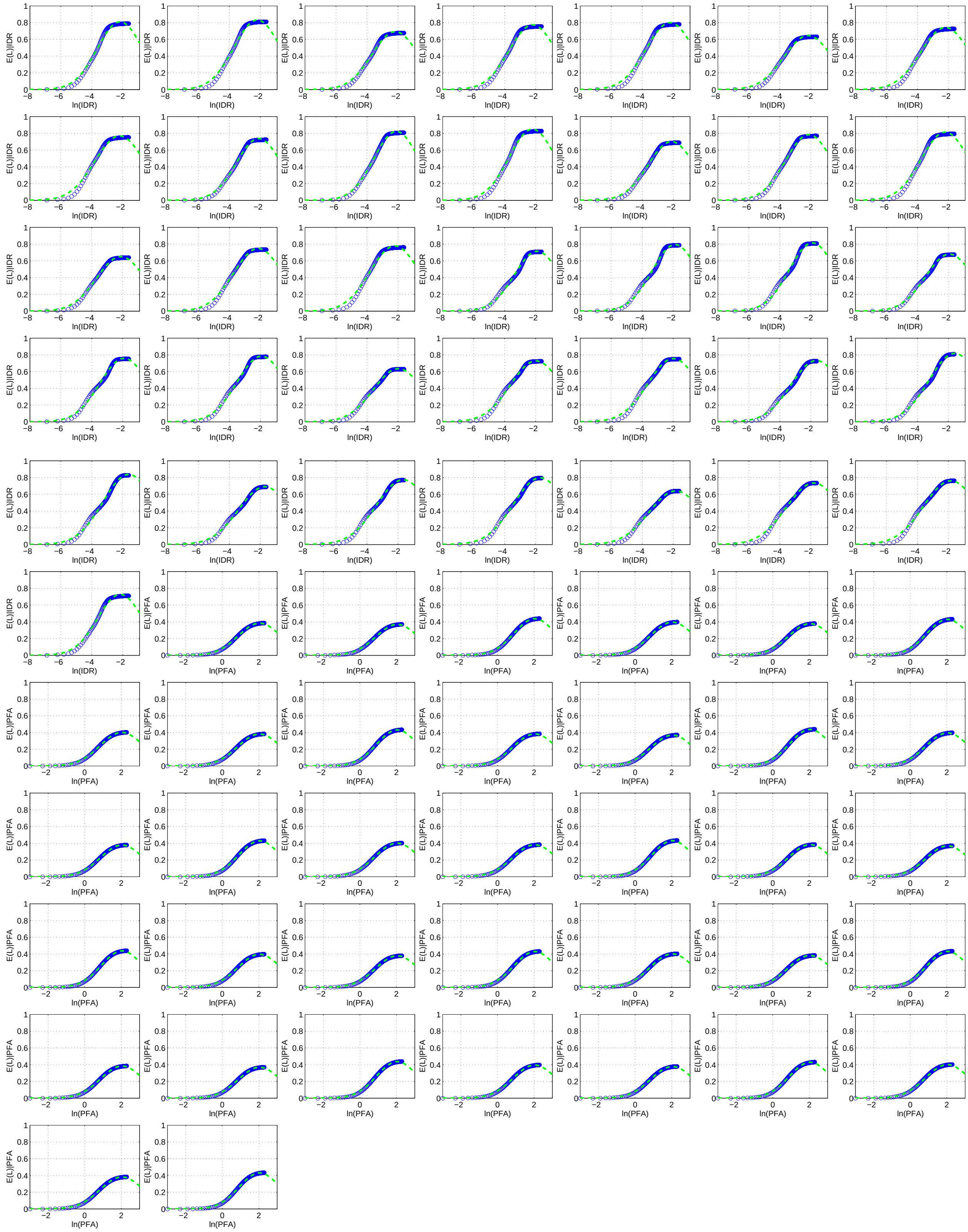

Figure 2. Fits of the proposed analytical model to IDR-based and PFA-based storey-based datasets available in [15]

As shown in Fig. 2, the adequate fit obtained with the proposed analytical model allows for a direct calculation of $E[L|n_{gm}]$ by summing the values of L|IDR and L|PFA from all the storeys of the building. The presented EDP-to-Loss functions were defined for a specific configuration of RC office buildings. Therefore, its applicability to other types of RC building cannot be assumed, since it may result in correlations that have different shapes due to the different architectural configurations of the storeys.

A simple approach to develop alternative functions was proposed by Ramirez *et al.* [16], using fragility and consequence models derived from HAZUS [17] generic data. To demonstrate that this strategy is still compatible with the proposed analytical model, an application was done to a residential RC frame building. The corresponding damage to loss model is shown in Fig. 3.

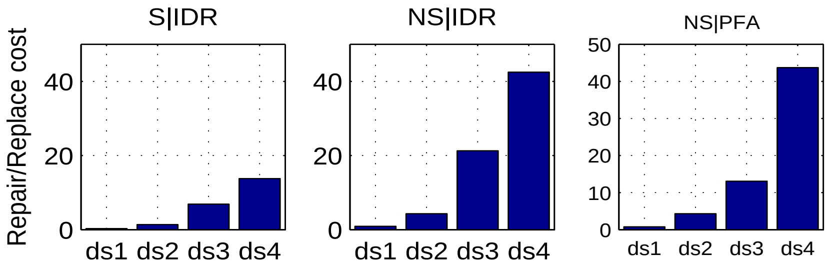

Figure 3. Damage-to-Loss model selected for a multi-storey residential building

The consequence model can be combined with the corresponding fragility models adopted for a CM4 structural typology as defined in HAZUS [17], yielding by simulation the correlations presented in Fig. 4.

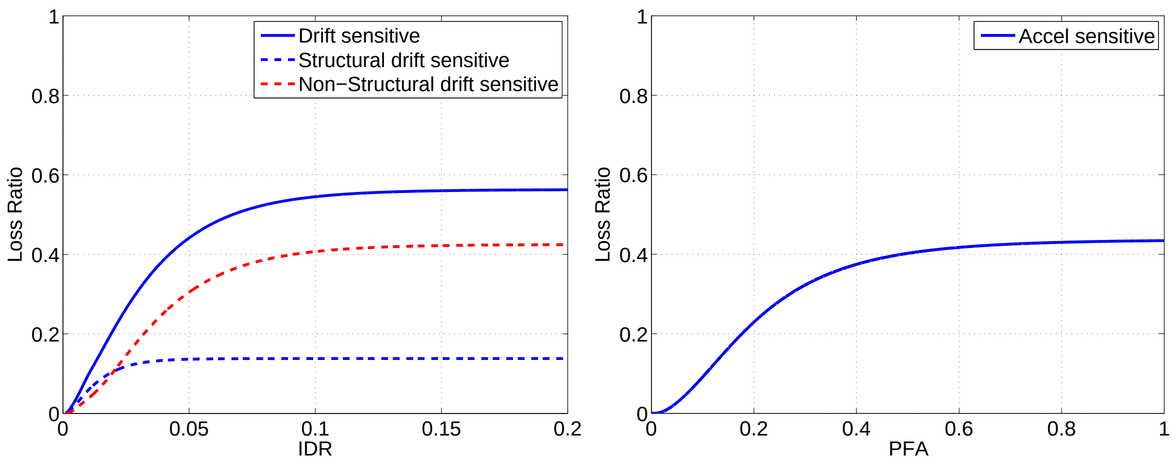

Figure 4. Storey-based EDP-to-Loss functions obtained using the HAZUS methodology.

Following the results provided by Martins *et al.* [18], the structural and non-structural losses associated with IDR can be combined. Accordingly, a weight of 77% is attributed to the IDR-sensitive component losses (L|IDR) resulting from the sum of 32% being assigned to the structural components (S|IDR) and 45% to the non-structural counterpart (NS|IDR). Finally, following the same component aggregation for a residential building in Portugal, 23% of the global storey losses are attributed to the repair needs of PFA sensitive components (L|PFA). Using these weights, the curves of Fig. 4 can be re-scaled yielding the curves and the corresponding analytical fit shown in Fig. 5.

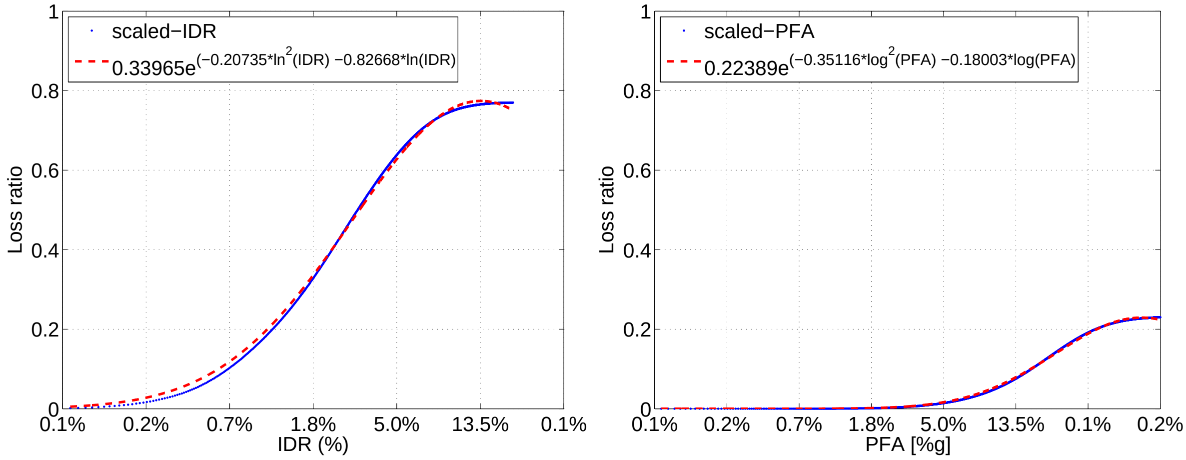

Figure 5. Fits of the proposed analytical model to IDR-based and PFA-based data derived from the HAZUS strategy.

As shown in the Fig. 5, the analytical function proposed in Eq. (11) is also seen to be an adequate fit to the data generated using the alternative method. As said before, these functions provide a faster way of computing the expected value of the losses for each storey and for each ground motion analysed. Moreover, it provides a significant advantage when adopting a risk-based assessment, since it can be directly included into the IDA stopping rule to be used in the calculation of the IM_{LS} distribution, as shown in the following section.

4. An alternative approach using a risk-based limit state approach

The procedure proposed in Section 3 provides a practice-oriented strategy consistent with the approach followed by traditional standards. Nevertheless, it is based on a single intensity assessment, and therefore fails to evaluate the effect of other intensities on the evaluation of the limit state condition. A possible alternative can be adopted if instead of adopting a single intensity on the assessment for which the limit states are verified, the mean annual frequency (MAF) of exceeding the formulated limit state capacity is adopted. Accordingly, exceedance rates in a given period of time are adopted as limit state conditions instead of verifying directly the limit state compliance criteria.

The adequacy of adopting this approach as opposed to multiple intensity-based assessments relies on the possibility of introducing information from multiple ground motion intensities in the same result. This strategy has been also incorporated in the CNR guidelines [10; 19] which formed the basis of the framework proposed hereon. The main assessment condition that has to be verified for each limit state requires defining two fundamental values: a capacity limit defining the minimum level of protection ($\lambda_{LS,C}$), and the corresponding demand ($\lambda_{LS,D}$), which is defined in terms of the MAF. Accordingly, the limit state LS is verified if Eq. (12) holds, i.e. if the MAF of exceeding a given limit state condition is lower than the adopted maximum risk threshold.

$$\lambda_{LS,D} < \lambda_{LS,C} \tag{12}$$

As in the previous section, the main tasks involved in the derivation of the proposed procedure refer to the quantification of $\lambda_{LS,C}$ in agreement with current standard principles, namely those presented in the EC8/3, and with the definition of an adequate procedure to quantify $\lambda_{LS,D}$ using analytical approximations and including information about the expected losses.

The levels of protection associated to each limit state provided in the EC8/3 are characterized by the selection of different return periods for the seismic action. For NC, an average return period (T_r) of 2475 years is suggested. For the limit state of SD, a T_r of 475 years is recommended while for the limit state of DL an average return period T_r equal to 225 years is proposed. These values refer to indicative proposals that can be changed in national annexes. Using these principles, Pinto and Franchin [10] described a set of minimum protection levels ($\lambda_{LS,C}$) represented by the maximum admissible seismic risk. The referred values are presented in Table 3 and were adopted also in the present framework. These seismic risk thresholds refer to the application of the simplified expression derived from the original SAC-FEMA approach [20], according to which $\lambda_{LS,C}$=2.25 ⁄ T_r.

Table 3. Maximum values of $\lambda_{LS,C}$ to ensure a similar level of protection as that currently required by the EC8/3.

Limit State	T_r (years)	Class I	Class II	Class III	Class IV
Damage Limitation, DL	225	0.0149	0.0100	0.0067	0.0049
Significant Damage, SD	475	0.0071	0.0047	0.0032	0.0023
Near Collapse, NC	2475	0.0014	0.0009	0.0006	0.0004

In the proposed methodology, the value of $\lambda_{LS,D}$ that has to be compared with the previous minimum protection levels can be analytically computed using the SAC-FEMA method with the quadratic approximation for the seismic hazard curve as proposed by Vamvatsikos [21]. Accordingly, the $\lambda_{LS,D}$ is given by:

$$\lambda_{LS,D} = \sqrt{p} \cdot k_0^{(1-q)} \cdot \left[H\left(\hat{s} \,|\, [Y_{LS}=1] \right) \right]^q \cdot exp\left[\frac{k_1^2}{4k_2} \cdot (1-q) \right] \tag{13}$$

where k_0, k_1 and k_2 are parameters of the seismic hazard function H(·) obtained from the fitting of a quadratic model in the logarithmic space to the hazard data of the site:

$$H\left(\hat{s} \,|\, [Y_{LS}=1] \right) = k_0 \cdot exp\left(-k_2 \cdot ln^2\left(\hat{s} \,|\, [Y_{LS}=1] \right) - k_1 \cdot ln\left(\hat{s} \,|\, [Y_{LS}=1] \right) \right) \tag{14}$$

with $\left(\hat{s} \,|\, [Y_{LS}=1] \right)$ representing the median IM level at which the scalar damage parameter Y_{LS} is equal to 1 and q is an uncertainty factor that depends on the corresponding dispersion $\beta_{s|Y_{LS}=1}$ and is defined by:

$$q = \frac{1}{1 + 2 \cdot k_2 \cdot \left(\beta_{s|Y_{LS}=1} \right)^2} \tag{15}$$

The SAC-FEMA method uses the results obtained from multiple ground motion records to compute the distribution of the IMs at which the limit state condition is verified. This so-called IM-based approach can be used in combination with any limit state condition involving the previously defined scalar loss variable, and can rely on the evaluation of this EDP obtained from incremental dynamic analysis (IDA), [22].

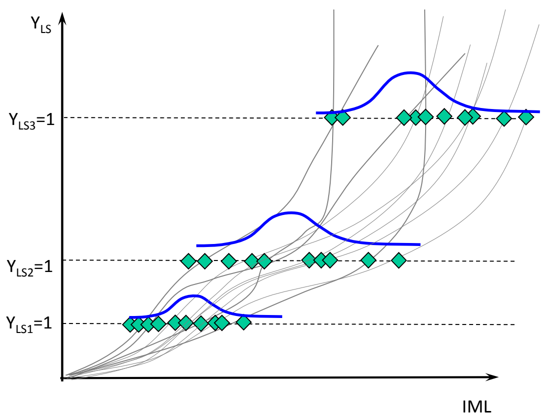

Figure 6. Illustrative representation of the IM-based distributions obtained for different LSs defined according to Eq. (9).

Therefore, $\left(\hat{s} \mid \left[Y_{LS}=1\right]\right)$ and $\beta_{s|Y_{LS}=1}$ can be computed using what is hereon designated as a Loss-based IDA methodology, based on the original IM-based IDA strategy proposed in [22]. Accordingly, a set of records must be scaled until the stopping rule Y_{LS}=1.0 is achieved, and the corresponding IM level recorded. The statistical treatment of the IM levels obtained for each ground motion corresponding to the unitary value of Eq. (9) can therefore be calculated, as illustrated in Fig. 6. Therefore, the proposed criterion (Eq. (10)) combined with the analytical approximations to compute $E[L|n_{gm}]$ allows for a direct evaluation of the IM_{LS} distribution since the loss-based limit state criteria proposed can be directly used inside the IDA.

5. Conclusions

The current European code for the seismic safety assessment of buildings, the Eurocode 8 –Part 3 has been analysed and criticized in the recent years because of several inconsistencies found in its assessment framework. Among others, the incompatibility between the performance objectives and the compliance criteria has been found to be one of the major issues. The present paper analysed the code definitions of performance objectives and evaluated their correlation with loss-based assessment methodologies such as the PEER-PBEE methodology. It was seen that the qualitative description of the performance objectives can be completely described by modern PBEE methods, particularly regarding the inclusion of the probability of demolition and the adoption of economic decision variables.

A practice-oriented methodology that includes losses in the seismic safety assessment of RC frame buildings has been calibrated using a scalar loss variable (Y_{LS}). Compliance criteria based on Y_{LS} were established in agreement with the performance objectives required by EC8/3. These compliance criteria were defined for the DL and SD limit states using Y_{LS} as the ratio between the observed and a maximum admissible cost. Specific limits for the admissible losses were proposed, including the damage limitation condition for non-structural components for the limit state of DL (loss limit of 0.10) and the uneconomic reparability condition for the limit state of SD (loss limit of 0.40). The former was defined based on recent recommendations according to which for repair costs above the proposed value stakeholders are keener to demolish a building instead of repairing it. The method proposed to quantify the observed losses relies on the explicit inclusion of the losses corresponding to repair needs and to the probability of demolition obtained based on the seismic demand. The method also allows for an implicit inclusion of the losses associated with structural collapse. Since one of the more complex tasks in quantifying Y_{LS} consists in estimating the repair losses, analytical models have also been proposed featuring a polynomial expression that was seen to provide an adequate to fit the data derived by Ramirez and Miranda [15]. An alternative approach to derive these functions was also tested, and the adequacy of the proposed mathematical model was also shown for that case.

One of the main advantages of having a full analytical condition based on a storey-based loss assessment methodology is that it allows for the definition of an alternative risk-based approach. This condition can be used as a stopping rule in the IDA strategy and used to quantify the distribution of the IM values associated with a given limit state. Using the SAC/FEMA approach, the proposed method allows for the calculation of the seismic risk and its comparison with the corresponding maximum admissible value calculated using the average return period of the seismic action defined in standards for each limit state.

The proposed methods introduce a clear and explicit compatibility between the performance objectives described in the current version of EC8/3, establishing a practice oriented approach that includes economic decision variables in the definition and verification process of standard-based limit states.

5. References

[1] SEAOC Vision 2000 Committee (1995): Performance-based seismic engineering. SEAOC, Sacramento, CA

[2] American Society of Civil Engineers (2000): Pre-standard and commentary for the seismic rehabilitation of buildings, Report No. FEMA-356, Washington, D.C.

[3] CEN (2005): ENV 1998-3. Eurocode 8: Design of structures for earthquake resistance - Part 3: Assessment and retrofitting of buildings. European Committee for Standardization, Brussels, Belgium.

[4] Bradley B (2013): A comparison of intensity-based demand distributions and the seismic demand hazard for seismic performance assessment. *Earthquake Engineering & Structural Dynamics* 42(15), 2235-2253.

[5] Lee T, Mosalam K (2006): Probabilistic seismic evaluation of reinforced concrete structural components and systems. *Technical Report PEER 2006/04*, Pacific Earthquake Engineering Research, Berkeley, USA

[6] Cornell C, Krawinkler H (2000): Progress and challenges in seismic performance assessment, *PEER Centre News* 3(2), 1-3.

[7] Ramirez C, Miranda E (2012): Significance of residual drifts in building earthquake loss estimation. *Earthquake Engineering & Structural Dynamics*, 41, 1477–1493.

[8] Aslani H (2005): Probabilistic earthquake loss estimation and loss disaggregation in buildings. *PhD Dissertation*. Stanford University.

[9] Franchin P, Pinto P, Rajeev P (2010): Confidence factor? *Journal of Earthquake Engineering* 14 (7), 989–1007.

[10] Pinto P, Franchin P (2014): Existing buildings: the New Italian provisions for probabilistic seismic assessment in *Perspectives on European Earthquake Engineering and Seismology*, 97-130.

[11] Romão X, Delgado R, Guedes J, Costa A (2010): A comparative application of different EC8-3 procedures for the seismic safety assessment of existing structures. *Bulletin of Earthquake Engineering* 8 (1), 91-118.

[12] Jalayer F, Franchin P, Pinto P (2007): A scalar damage measure for seismic reliability analysis of RC frames. *Earthquake Engineering & Structural Dynamics* 36(13), 2059-2079.

[13] FEMA (2012) Next-Generation Methodology for Seismic Performance Assessment of Buildings, *Report No. FEMA P-58*, Washington, D.C.

[14] Zareian F, Krawinkler K (2006): Simplified performance-based earthquake engineering. *Report No. 169. John A. Blume Earthquake Engineering Research Centre*. Stanford University. Stanford, California.

[15] Ramirez C, Miranda E (2009): Building-specific loss estimation methods & tools for simplified performance-based earthquake engineering. *Report No. 171. John A. Blume Earthquake Engineering Research Centre*. Stanford University. Stanford, California.

[16] Ramirez C, Gupta A, Myers A (2012): Detailed seismic loss estimation for a tall building in Japan. *15th World Conference on Earthquake Engineering*, Lisbon, Portugal.

[17] FEMA (2003): *HAZUS-MH MR5, Technical manual*, Department of Homeland Security—Federal Emergency Management Agency.

[18] Martins L, Silva V, Marques M, Crowley H, Delgado R (2015): Development and assessment of damage-to-loss models for moment-frame reinforced concrete buildings, *Earthquake Engineering & Structural Dynamics*, 45(5), 797-817.

[19] CNR (2014): CNR DT 212/2013. Istruzioni per la Valutazione Affidabilistica della Sicurezza Sismica degli Edifici Esistent. Consiglio Nazionalle delle Ricerche – Commisione de studio per la predisposizione e l'analisi di norme technique relative alle construzioni. Roma. May, 2014. (*in Italian*).

[20] Cornell C, Jalayer F, Hamburger R, Foutch D (2002): Probabilistic basis for 2000 SAC Federal Emergency Management Agency steel moment frame guidelines. *Journal of Structural Engineering*, 128:526–533.

[21] Vamvatsikos D (2013): Derivation of new SAC/FEMA performance evaluation solutions with second-order hazard approximation. *Earthquake Engineering & Structural Dynamics*, 42 (8) 1171–1188.

[22] Vamvatsikos D, Cornell C (2002): Incremental dynamic analysis. *Earthquake Engineering & Structural Dynamics*, 31 (3), 491-514.