Envelope-based pushover analysis procedure for assessing the collapse risk of buildings

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ABSTRACT: The pushover-based procedure for assessing the collapse risk of taller buildings is proposed. The procedure utilizes recently introduced envelope-based pushover analysis (EPA), which involves so-called modal-based and failure-based SDOF models corresponding to three pushover analyses. The EPA procedure is used to determine the collapse fragility, whereas the assessment of collapse risk is based on the well-known closed-form solution of the risk equation. The procedure is demonstrated by means of an example of a 15-storey reinforced concrete frame building taking into account two hazard-consistent sets of ground motions. It is shown that the EPA procedure provided sufficiently accurate estimates of collapse risk in comparison to that based on the nonlinear response history analysis. The basic pushover method, which is not capable of simulating different system failure modes due to the effects of ground motions, failed to predict the collapse risk with a useful degree of accuracy.

1 INTRODUCTION

The collapse risk assessment of buildings combines seismic hazard analysis, assessment of structural vulnerability and treatment of uncertainties. From theoretical point of view, the most accurate approach to assess vulnerability of structure is based on nonlinear response history analysis. This analysis is computationally demanding and becomes highly uncertain for predicting collapse capacity of realistic buildings. Consequently, many simplified nonlinear procedures for seismic performance assessment of buildings have been developed in last decades, which usually combine nonlinear static (pushover) analysis of entire structure and response spectrum analysis (RSA) or nonlinear response history analysis (NRHA) of an equivalent single-degree-of-freedom (SDOF) model.

Many variants of the basic pushover-based methods have been proposed in order to improve their ability for prediction of seismic response of structures and to overcome a basic assumption, i.e. that the structures vibrates predominantly in a single mode. For example, modal pushover analysis (MPA) procedure was introduced by Chopra & Goel (2002) in order to adequately address the issue of higher mode effects. The procedure involves several pushover analyses, which are based on invariant distributions of lateral forces and appropriate combination of results in order to determine the total seismic demand. The basic MPA was recently extended (Reyes & Chopra 2011) to the three-dimensional analysis of buildings subjected to two horizontal components of ground motions. Kreslin & Fajfar (2011) proposed the extended N2 method, which takes into account the effects of higher modes by enveloping the results determined with the basic N2 method (Fajfar 2000) and RSA. Additionally, Sucuoğlu and Günay (2011) proposed multi-mode pushover analysis by using generalized force vectors, which does not require combinations of results or correction factors. Authors of the above-mentioned procedures have shown that the simplified procedures for performance assessment of buildings are able to provide fair estimates in the case of the test examples.

However, Bobadilla & Chopra (2008) have shown that the 'first-mode' SDOF model provides biased estimates of roof displacement for individual ground motions (Bobadilla and Chopra, 2008). In order to overcome this issue, Brozovič and Dolšek (2013) proposed so-called envelope-based pushover analysis (EPA) procedure, which enables prediction of the seismic demand for a particular ground motion at specific intensity level by enveloping the results associated with three pushover analyses. EPA assumes that each response parameter is controlled by the predominant system failure mode, which can be detected by introducing so-called failure-based SDOF models.

In this paper, the EPA procedure is briefly described and demonstrated by assessing the collapse risk of fifteen-storey reinforced concrete frame building taking into account two sets of hazard-consistent ground motions (Baker 2011a). The collapse risk is estimated on the basis of closed-form solution of the risk equation (Cornel 1996, McGuire 2004). The collapse risk based on EPA procedure is compared with the collapse risk obtained by the basic pushover-based method, which uses only the 'first-mode' pushover analysis (PA1), and the nonlinear response history analysis (NRHA).

2 OVERVIEW OF ENVELOPE-BASED PUSHOVER ANALYSIS AND RISK ASSESSMENT PROCEDURE

2.1 *Overview of the envelope-based pushover analysis procedure*

2.1.1 *Pushover analyses*

Pushover analyses are performed by using an invariant horizontal load distribution $\boldsymbol{s}_i$, which corresponds to the product of the diagonal mass matrix $\boldsymbol{M}$ and the ith natural vibration mode $\boldsymbol{\phi}_i$

$$\boldsymbol{s}_i = \boldsymbol{M} \cdot \boldsymbol{\phi}_i \tag{1}$$

This is the most common approach, which is consistent with the basic pushover-based methods (e.g. Fajfar 2000, Vamvatsikos & Cornell 2006, Peruš et al. 2013), which involve only the first vibration mode distribution of lateral forces (i = 1). However, the second and third vibration mode distributions of lateral forces (i = 2, 3) are also used by analogy with the MPA procedure (Chopra & Goel 2002).

Results of the pushover analyses are three pushover curves, which are idealized in order to define the force-displacement relationships of the equivalent SDOF models. Based on the results of each pushover analysis, storey drifts and the estimated level of damage to individual structural elements can be directly assessed for a given target roof displacement, which is determined by using an appropriate single-degree-of-freedom (SDOF) model.

2.1.2 *Modal-based and failure-based SDOF models*

The EPA procedure involves so-called failure-based SDOF models, which are intended to be used for predicting the roof displacement in the case if ground motions cause system failure modes, which are significantly different to that observed for the 'first-mode' pushover analysis (Brozovič & Dolšek 2013).

The failure-based SDOF models were defined since comparative studies have shown that modal-based SDOF models may not estimate target roof displacement to a usefully degree of accuracy for an individual ground motion (Bobadilla & Chopra 2008). In fact, the displacement demand of modal-based SDOF models is almost always governed by the first vibration mode.

According to basic pushover-based method (e.g. Fajfar 2000) the force-displacement relationships of the so called modal-based SDOF models (F^* - D^*) are determined by dividing the base shear F and roof displacement D of the idealized pushover curves by a transformation factor Γ. By analogy with MPA (Chopra & Goel 2002), this factor is defined as follows

$$\Gamma_{m,i} = \frac{m^*_{m,i}}{\sum_{j=1}^{n} m_j \phi^2_{i,j}}, \quad m^*_{m,i} = \sum_{j=1}^{n} m_j \phi_{i,j} \tag{2}$$

where m_j is the jth storey mass, $\phi_{i,j}$ is the jth component of the ith natural vibration mode, which, in this paper, is always normalized to a roof displacement equal to 1, and $m^*_{m,i}$ is the mass of the modal-based SDOF model corresponding to the ith natural vibration mode. This is the commonly used definition of the transformation factor and mass of the SDOF model, which are herein denoted by the index m, since $\Gamma_{m,i}$ is in fact the modal participation factor for the ith natural vibration mode. The period of the ith modal-based SDOF model is defined as follows

$$T^*_{m,i} = 2\pi \sqrt{\frac{m^*_{m,i} D_{1,i}}{F_{1,i}}} \tag{3}$$

where $D_{1,i}$ and $F_{1,i}$ correspond to the roof displacement and base shear at point in linear elastic range of the idealized 'ith-mode' pushover curve, respectively.

Definition of the parameters of the failure-based SDOF models is slightly different. The transformation factors and equivalent SDOF masses of the failure-based SDOF models are based on the displacement vectors corresponding to the system failure modes which are observed from pushover analyses, and are defined as follows

$$\Gamma_{f,i} = \frac{m^*_{f,i}}{\sum_{j=1}^{n} m_j d^2_{i,j}}, \quad m^*_{f,i} = \sum_{j=1}^{n} m_j d_{i,j} \tag{4}$$

where $d_{i,j}$ is the jth component of the normalized 'ith failure-mode' displacement vector ($d_{i,n}$ = 1) associated with the pushover analysis for the 'ith vibration mode' force distribution. Consequently, the period of the ith failure-based SDOF model

$$T^*_{f,i} = 2\pi \sqrt{\frac{m^*_{f,i} D_{1,i}}{F_{1,i}}} \tag{5}$$

can differ slightly in comparison with $T^*_{m,i}$, if a difference between $m^*_{f,i}$ and $m^*_{m,i}$ occurs.

According to Brozovič and Dolšek (2013) the system failure mode is associated with 80% strength in the softening branch of the 'ith-mode' pushover curve. It should be noted that the normalized failure-mode displacement vector, which correspond to the 'first-mode' pushover curve, is very similar to the first natural vibration mode. For this reason modal-based and failure-based SDOF models are almost equal when they correspond to the 'first-mode' pushover curve since $\Gamma_{f,1} \approx \Gamma_{m,1}$.

The ability of the equivalent SDOF models to predict the roof displacement also depends on their hysteretic and damping model. In general the SDOF model should simulate the global response of a structure and thus depends on the type of structure which is the subject of a seismic performance assessment. In this paper the peak-oriented model is used to describe the hysteretic behaviour of the SDOF model (Peruš et al. 2013). Note that it has already been shown that such a model is sufficiently accurate for the simulation of the global cyclic behaviour of reinforced concrete buildings (e.g. Fajfar et al. 2006), provided that monotonic curves implicitly account for cyclic strength deterioration. In the EPA procedure it is assumed that the damping ratio is the same for all three SDOF models. It should be noted that this is not consistent with the modal response history analysis (e.g. Chopra 2007). However, parametric studies have shown that it is likely that the accuracy of the overall results of the EPA procedure would be reduced if the failure-based SDOF models were based on different assumptions (Brozovič & Dolšek 2013).

2.1.3 *Prediction of collapse capacity by using EPA procedure*

The estimation of the collapse capacity for an individual ground motion according to EPA procedure involves the following steps:

1) Perform pushover analyses for first, second and third mode distributions of lateral forces and determine corresponding idealized force-displacement relationships.
2) Determine the modal-based SDOF model corresponding to the 'first-mode' pushover curve ($\Gamma_{m,1}$, $m^*_{m,1}$, $T^*_{m,1}$) and two failure-based SDOF models corresponding to the 'second-mode' ($\Gamma_{f,2}$, $m^*_{f,2}$, $T^*_{f,2}$) and 'third-mode' pushover curves ($\Gamma_{f,3}$, $m^*_{f,3}$, $T^*_{f,3}$).
3) Determine the collapse capacity for the modal-based and each failure-based SDOF model by using the response history analysis. The collapse capacity could be achieved on the basis of bisection by scaling the ground motion intensity until collapse capacity is achieved with predetermined accuracy.
4) The collapse capacity, expressed in terms of ground-motion intensity, is obtained by enveloping results associated with the three SDOF models.

Such procedure assumes that the collapse capacity is controlled by the predominant system failure mode caused by a ground motion. In more general case the EPA procedure can be used for the intensity-based seismic performance assessment. In this case the total seismic demand for each response parameter and for a particular ground motion is determined by enveloping the results associated with each pushover analysis (Brozovič & Dolšek 2013). It should be noted that such an approach does not require any combination of results or correction factors.

However, Brozovič & Dolšek (2013) have discussed that invariant force-based pushover analysis may not be appropriate for the simulation of all system failure modes which were observed from response history analysis. Namely, the damage in the buildings was in the case of some ground motions concentrated in the top part as well as in the lower part of the building, and also varied with respect to the level of intensity. Even in these cases the EPA procedure proved to be sufficiently accurate or at least provided conservative estimates for the collapse capacity. More details regarding the EPA are available elsewhere (Brozovič & Dolšek 2013). The EPA procedure is also a component of the risk-based design procedure as demonstrated in the paper presented in this conference (Lazar & Dolšek 2013).

2.2 *Risk assessment procedure*

In the case study presented in this paper the seismic risk was estimated in terms of mean annual frequency of collapse λ_C. Cornell (1996) and McGuire (2004) have shown that λ_C can be expressed in closed-form as follows

$$\lambda_C = H(S_{a,C}) \cdot e^{0.5 \cdot k_C^2 \cdot \beta_C^2} \tag{6}$$

where $S_{a,C}$ is the median value of the IM-based collapse capacity (i.e. the median spectral acceleration at fundamental period which causes collapse), β_C is corresponding standard deviation of natural logarithms, k_C is the slope of seismic hazard function in log domain, and $H(S_a)$ is the hazard function, i.e. the mean annual frequency of exceedance of intensity S_a.

The value of the collapse capacity, which is determined according to the procedure described in Section 2.1.3, varies from record-to-record and, more generally, due to the effect of epistemic uncertainties. For simplicity reasons, only record-to-record randomness is herein taken into account for estimation of the parameters $S_{a,C}$ and β_C. Since it has been assumed that the spectral acceleration at fundamental period is distributed log-normally, the fragility parameters $S_{a,C}$ and β_C are estimated according to the method of moments, as follows

$$S_{a,C} = \overline{S}_{a,C} \cdot e^{-0.5 \beta_C^2} \tag{7}$$

$$\beta_C = \sqrt{\ln\left(\frac{\sigma^2}{\overline{S}_{a,C}^2}+1\right)} \tag{8}$$

where $\overline{S}_{a,C}$ and σ^2 are, respectively, the sample mean of collapse capacity and corresponding variance, which are obtained for a set of hazard-consistent ground motions.

3 EXAMPLE: ASSESSMENT OF SEISMIC COLLAPSE RISK FOR A FIFTEEN-STOREY REINFORCED CONCRETE FRAME BUILDING

The assessment of seismic collapse risk is performed for a fifteen-storey reinforced concrete frame building. The fragility parameters were determined by the envelope-based pushover analysis and for comparison reasons also by nonlinear response history analysis (NRHA) and basic pushover-based procedure (PA1), where 'first-mode' distribution of lateral forces is used. In the later case the collapse capacity is assessed only on the basis of modal-based SDOF model corresponding to 'first-mode' pushover curve. The spectral acceleration at fundamental period $S_a(T_1)$ and the maximum storey drift ratio θ_{max} are adopted, respectively, as the intensity measure and the engineering demand parameter.

3.1 *Description of the structure and mathematical model*

Fifteen-storey reinforced concrete frame building, which is sensitive to system-failure-mode effects due to ground motions, is selected as an example structure. It is designed as earthquake resistant structure in compliance with Eurocode 8 (CEN 2004) for design ground acceleration of 0.25 g, soil class B, ductility class medium and behaviour factor of 3.9.

Elevation and plan view of the building is shown in Figure 1. It can be observed that the structure is symmetric in plane in the indicated direction of seismic loading. Cross-section dimensions of columns are decreasing with elevation and amount to 70/80 cm from 1st to 9th storey, 60/70 cm from 10th to 12th storey and 50/60 cm in upper stories. All the beams have dimensions of 55/60 cm, while slabs are 22 cm thick. The steel S500 is used for reinforcement, whereas the concrete quality is reducing with elevation of the building, i.e. from C40/50 in the first and second storey to C25/30 at the top storeys. For details see Brozovič & Dolšek (2013).

A simplified nonlinear model was built using PBEE toolbox (Dolsek 2010) in conjunction with OpenSees (2011). The mass and mass moment of inertia were concentrated in the centre of gravity of each floor diaphragms, which were assumed to be rigid in their own planes. Effective width of the beams is modelled according to Eurocode 2 (CEN 2004), assuming zero moment points at the mid-span of the beams.

Beam and column flexural behaviour is modelled by one-component lumped plasticity elements, consisting of an elastic beam and two inelastic rotational hinges. Note, that the moment-rotation relationship was determined according to PBEE toolbox (Dolsek 2010). However, the trilinear moment-rotation relationship is schematically presented later (Figure 4).

Cyclic behaviour in the plastic hinges of beams and columns was simulated by using uniaxial hysteretic material available in OpenSees (2011). Unloading stiffness was defined by parameter β_U, which is assumed to have a value of 0.8. Effect of pinching was not simulated. The deterioration under cyclic deformations was implicitly accounted for by the moment-rotation envelopes of plastic hinges. This type of simplified nonlinear model of a structure was validated several times against the experimental results from the full-scale pseudo-dynamic tests (e.g. Fajfar et al. 2006, Dolsek 2010).

Pushover or response history analyses were performed after the gravity load had been simulated. P-Delta effects were considered for all analyses. Mass proportional damping (5% damping ratio at the fundamental period of vibration) was used in the response history analyses. It provided slightly greater collapse capacity values if compared to those obtained by assuming damping proportional to instantaneous stiffness.

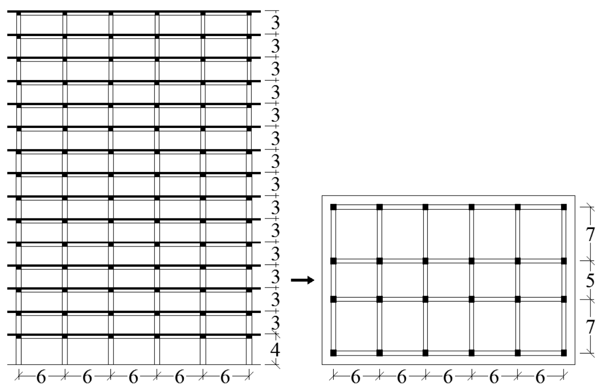

Figure 1. Elevation and plan view of the building with indicated direction of seismic loading.

3.2 *Sets of ground motions*

Two hazard-consistent sets of forty ground motions were selected in order to match the target spectra for 475-year and 10.000-year return periods.

The hazard-consistent ground motions were selected according to the concept of conditional spectrum (CS), which was proposed by Baker (2011a). A computationally efficient ground motion selection algorithm for matching the conditional spectrum mean and variance (Jayaram et al. 2011) was used, whereas the seismic hazard parameters needed for the definition of conditional spectra, were determined based on EZ-FRISK (Baker 2011b and EZ-

FRISK 2012) for site in Ljubljana, Slovenia. Note that conditioning period T_c was equal to fundamental period T_1 of the 15-storey building, which amounted to 1.9s. Target spectral acceleration at fundamental (conditioning) period has a value of 0.09g in case of 475-year return period and 0.42g in case of 10.000-year return period. The selected sets of records were named after their target spectra (set CS 475 and set CS 10.000). Conditional mean spectrum (CMS) with indicated conditional standard deviation, spectra of the individual ground motions and corresponding median spectrum are shown in Figure 2.

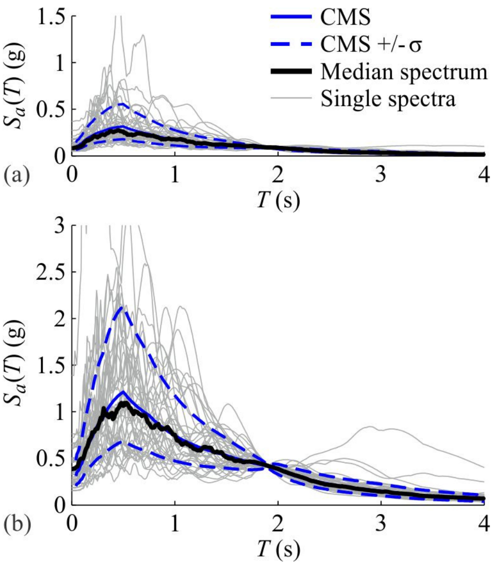

Figure 2. Acceleration spectra of ground motions, corresponding median spectrum and CMS with conditional standard deviation for (a) 475-year and (b) 10.000-year return period.

3.3 *Pushover analyses and the SDOF models*

Pushover analyses were performed for one direction (Figure 1) using the three invariant horizontal force distributions (Eq.(1)). The roof displacement was monitored, using an increment of 0.005 m.

According to pushover curves (Figure 3) it can be seen that the variation of maximum base shear was not large. Therefore, the maximum base shear versus weight of the building ratio (F_b/W) was similar for all three lateral force distributions and amounted to 7% for the 'first–mode' pushover analysis and 6% for the 'second-mode' and 'third-mode' pushover analysis.

The near-collapse (NC) limit state was assumed at the top displacement corresponding to 80% strength in the softening branch of the pushover curves (Figure 3). The corresponding damage is presented in Figure 4. Note that the grey, green, yellow and red colours represent the damage in the plastic hinges of columns and beams, as graphically described by the moment-rotation relationship as shown in Figure 4.

Three SDOF models were determined as defined in Section 2.1.2. Idealized base shear – roof displacement relationships, which were used for the determination of the force-displacement relationship of the equivalent SDOF models are presented in Figure 3. Based on such idealization it is achieved that the period T^* of the modal-based SDOF model coincide with the fundamental period T_1 of the building, since the initial stiffness of the modal-based SDOF model is set equal to that obtained from the 'first-mode' pushover analysis. Values of the periods T^* and equivalent masses m^* corresponding to the failure-based SDOF models were similar to values, which could be defined for modal-based SDOF models (Table 1). However, transformation factors (Γ) for failure-based SDOF models were quite large. Consequently peak (yield) accelerations of the capacity diagrams S_{ay} of the failure-based SDOF models were quite lower in comparison to those based on modal-based SDOF models. It should be noted, that transformation factors for all three SDOF models used in EPA were of similar order of magnitude, which would not be the case if only modal-based SDOF models were used. This is the main reason that the failure-based SDOF models enables approximate simulation of the global seismic demand associated with failure modes that cannot be simulated by the modal-based SDOF models.

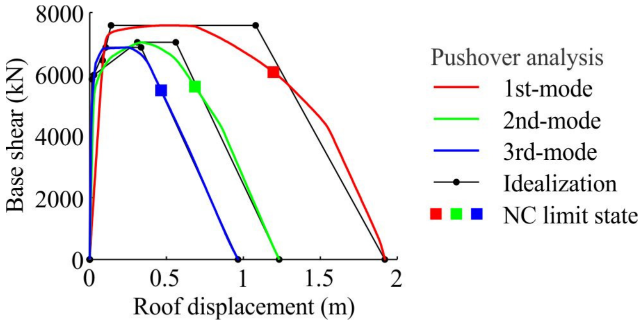

Figure 3. The 'first-mode', 'second-mode', 'third-mode' pushover curves and corresponding idealized base shear – roof displacement relationships. The highlighted points correspond to 80% strength in the softening branch of the pushover curves, indicated as the near collapse (NC) limit state.

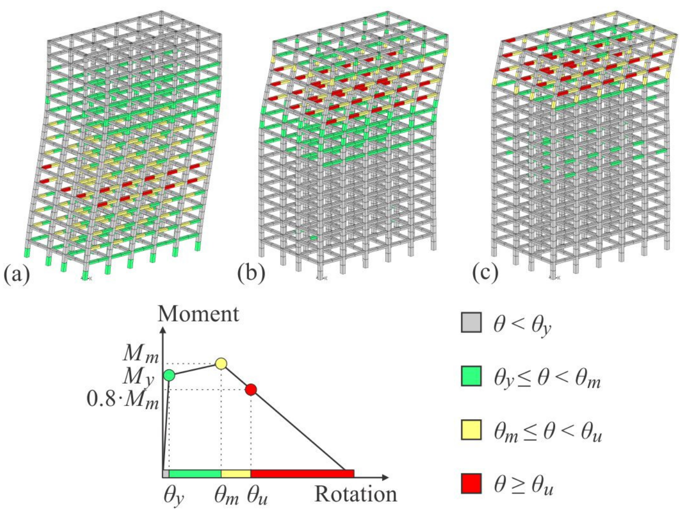

Figure 4. Deformed building at the NC limit state with indicated damage for (a) 'first-mode', (b) 'second-mode', (c) 'third-mode' pushover analysis and typical moment – rotation relationship for columns and beams.

Table 1. The periods T^*, masses m^*, transformation factors Γ and peak (yield) accelerations of the capacity diagrams S_{ay} of the SDOF models used in the EPA procedure.

Mode (type)	T^* (s)	m^* (t)	Γ	S_{ay} (g)
1 (modal-based)	1.90	7010	1.35	0.08
2 (failure-based)	0.72	2730	1.20	0.22
3 (failure-based)	0.39	1560	1.27	0.35

3.4 *Collapse capacity*

Collapse capacity was assessed as described in Section 2.1.3 and presented in terms of so-called IDA curves(Vamvatsikos & Cornell 2002). The results of the EPA procedure are compared with the results of the nonlinear response history analysis (NRHA) of entire structure and the basic pushover-based procedure, which involves 'first-mode' pushover analysis (PA1).

In the case of the approximate procedures (EPA and PA1), the seismic demand of the SDOF models was determined by the response history analysis. The peak ground acceleration, which caused global dynamic instability of the structural model, was estimated according to the bisection method with an error of less than 2% g. Each response history analysis at the structural level or at the level of the equivalent SDOF model was performed by using the Newmark integration scheme (γ_N = 0.5, β_N = 0.25) and assuming an integration time step of 0.01s.

3.4.1 *The 16th, 50th and 84th percentile IDA curves*

For illustration, the approximate percentile IDA curves based on approximate seismic response analyses (EPA and PA1) are compared with the "exact" IDA curves based on nonlinear response history analysis (Figure 5). Similar results were obtained for both sets of ground motion records (Section 3.2). Therefore results are presented only for ground motion set CS 10.000.

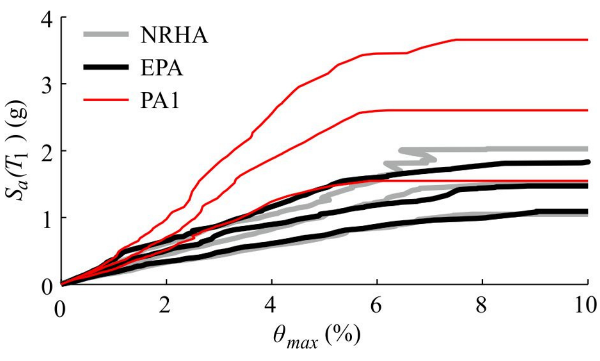

Figure 5. Comparison between the 16th, 50th and 84th percentile IDA curves obtained by using NRHA and the approximate procedures denoted as EPA and PA1 for ground motion set CS 10.000.

An excellent agreement can be observed between the approximate percentile IDA curves obtained by the EPA procedure and the 'exact' IDA curves, whereas the approximate percentile IDA curves obtained by the PA1 procedure significantly underestimate the seismic demand and consequently overestimate the collapse capacity.

3.4.2 *Predominant system failure modes according to EPA*

According to the results presented in Figure 5 it is clear that the fundamental system failure mode obtained by the 'first-mode' pushover analysis is not sufficient for simulation of seismic response of the building with a useful degree of accuracy. Therefore it was interesting to analyse for which ground motion and intensity levels the second and the third system-failure modes controlled the seismic response determined by EPA procedure. Results are presented in Figure 6 where it can be observed that the predominant system failure mode varies with respect to the intensity level and ground motion record. In this example, the second and third system failure modes represent predominant system failure modes for most of ground motions from the set corresponding to return period 10.000 years. Similar observations were made by analysing results obtained on the basis of ground motion set CS 475.

The influence of the failure-based SDOF models on collapse capacity of the building is presented in Figure 7. It can be observed that the collapse capacities $S_{a,C,\mathrm{PA1}}$ are not predicted with useful degree of accuracy for most ground motions, whereas failure-based SDOF models used by EPA procedure significantly improve prediction of the collapse capacities. However, EPA provides quite conservative estimates for collapse capacity corresponding to some ground motions. This issue is discussed elsewhere (Brozovič and Dolšek 2013).

3.4.3 *Influence of target return period on conditional spectrum and seismic response of the building*

Herein the hazard-consistent ground motion records were selected based on spectral shape and matching with conditional spectrum CS. Two ground motion sets were selected for two return periods (475 years and 10.000 years).

The shape of the normalized mean conditional spectrum (CMS) depends on the return period as it can be observed in Figure 8a. Significant difference appeared for the periods lower than the conditioning period (T= 1.9s). The peak acceleration corresponding to CMS for 10.000-year return period is quite lower in comparison to CMS for 475-year return period. However, the difference between the two spectra is negligible for periods larger than the conditioning period.

The difference in the spectra had an impact on the response of the structure (Figure 8b). The seismic demand for ground motion set CS 475 was greater than that corresponding to ground motion set CS 10.000. However, the difference in response increases for maximum storey drift ratios larger than

3%. It should be noted, that these observations were obtained on the basis of probabilistic seismic hazard analysis for Ljubljana (Slovenia) by using a simplified seismic model available in EZ-FRISK.

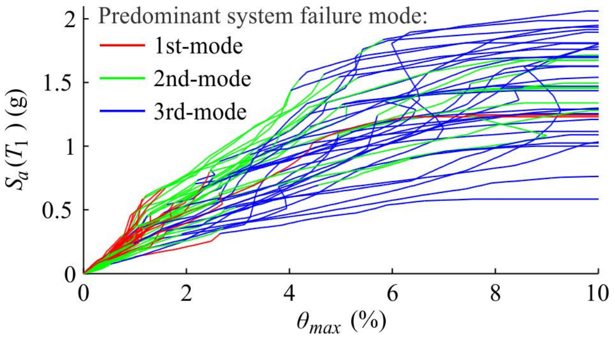

Figure 6. Approximate EPA-based IDA curves with indicated predominant system failure modes for ground motion set CS 10.000.

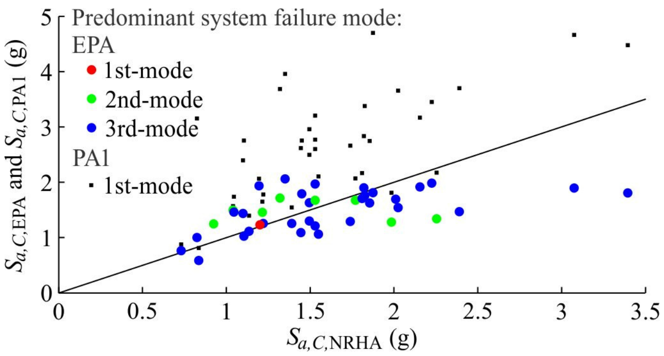

Figure 7. Collapse capacities determined by EPA and PA1 in comparison to NRHA-based collapse capacities for ground motion set CS 10.000. Different colors indicate predominant system failure modes.

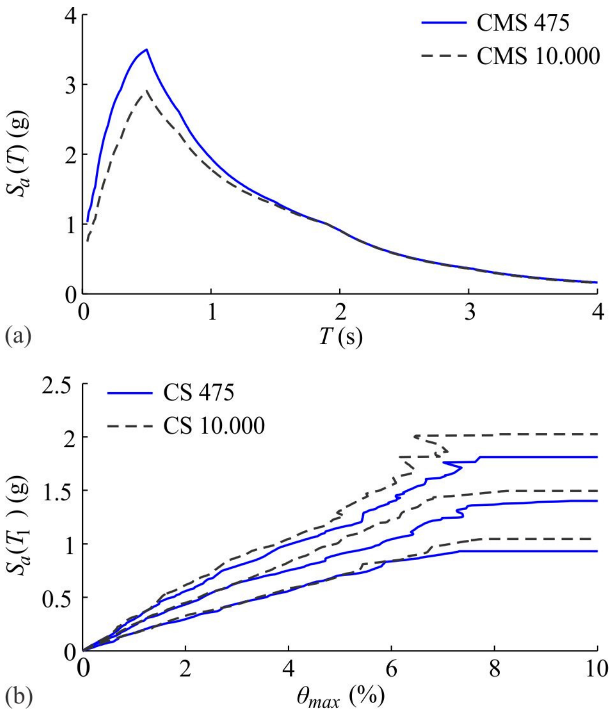

Figure 8. (a) CMS for 475-year and 10.000-year return periods for site in Ljubljana, Slovenia and (b) "exact" IDA curves for ground motion sets selected to match CS for 475-year and 10.000-year return period.

3.5 *Assessment of collapse risk*

Mean annual frequency of collapse λ_C was determined according to Eq.(6). The median collapse capacity $S_{a,C}$ and corresponding standard deviation of natural logarithms β_C, were assessed for both sets of ground motions by using EPA procedure. For comparison reasons the fragility parameters were obtained also by basic 'first-mode' pushover-based procedure (PA1) and NRHA (Table 2). It can be observed that the median collapse capacities $S_{a,C}$ determined by EPA accurately estimate NRHA-based values with a relative error of only 3%. However, EPA underestimates the values of standard deviation of natural logarithms β_C for 17% and 34% in comparison to "exact" NRHA-based results, respectively, for ground motion set CS 475 and CS 10.000. On the other hand, PA1 procedure highly overestimate fragility parameters, since $S_{a,C}$ values are more than 60% larger in comparison to those obtained by NRHA and β_C values are overestimated for 33% and 6%, respectively, for ground motion sets corresponding to 475-year and 10.000-year return periods.

The collapse risk was obtained by using the seismic hazard for Ljubljana, Slovenia (Baker 2011 and EZ-FRISK 2012) (Figure 9). Note, that the slope of an approximate hazard curve k_C = 2.24 was determined by using the method of least squares for the interval of spectral accelerations as presented in Figure 9.

Table 2. Fragility parameters (i.e. median collapse capacities $S_{a,C}$ and corresponding standard deviations of natural logarithms β_C) for the three seismic response analysis methods and both sets of ground motions.

	NRHA		EPA		PA1	
	$S_{a,C}$ (g)	β_C	$S_{a,C}$ (g)	β_C	$S_{a,C}$ (g)	β_C
CS 475	1.37	0.30	1.34	0.25	2.52	0.40
CS 10.000	1.50	0.35	1.45	0.23	2.39	0.37

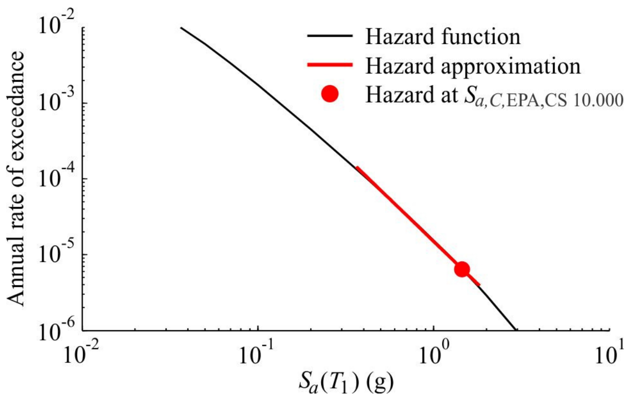

Figure 9. Seismic hazard function for central Slovenia (Ljubljana) and linear approximation of hazard function in log-log space.

The mean annual frequencies of collapse λ_C are presented in Table 3. It can be seen that EPA provided sufficiently accurate estimates of λ_C, whereas the λ_C determined on the basis of the PA1 is underestimated for more than 60%.

The collapse risk obtained by the ground-motion set CS 475 exceeded the collapse risk based on ground-motion set CS 10.000 for less than 25%.

Table 3. Mean annual frequency of collapse λ_C for three methods and two sets of ground motions.

GM set	NRHA	EPA	PA1
CS 475	$9.4\cdot10^{-6}$	$9.4\cdot10^{-6}$	$2.8\cdot10^{-6}$
CS 10.000	$8.1\cdot10^{-6}$	$7.6\cdot10^{-6}$	$3.0\cdot10^{-6}$

4 CONCLUSIONS

Recently proposed approximate seismic response procedure named envelope-based pushover analysis (EPA) was used for estimation of collapse risk. EPA is based on force-invariant pushover analyses and enables approximate simulation of those system failure modes, which cannot be simulated by using the 'first-mode' pushover analysis.

The ability of the envelope-based pushover analysis procedure to predict the collapse risk was demonstrated by means of an example of 15-storey reinforced concrete frame building. It was shown that for this building the basic pushover-based method provided inaccurate estimate of collapse risk, whereas results based on the EPA procedure were in excellent agreement in comparison to results based on the nonlinear response history analysis. Increased accuracy of the EPA procedure for predicting collapse risk is the consequence of so-called failure based SDOF models, which are defined in order to simulate different system failure modes, which are observed from response history analysis.

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