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PUSHOVER-BASED PROCEDURE FOR APPROXIMATE SIMULATION OF SYSTEM FAILURE MODES OBSERVED DUE TO GROUND MOTIONS

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ABSTRACT

Randomness of ground motions causes variability of system failure modes of a structure. Other parameters, which affect the formation of the failure modes, are the ground-motion intensity and sensitivity of the structure to variation of system failure mode. All these factors make it difficult to anticipate a system failure mode for a given ground motion without performing nonlinear response history analysis. Consequently, pushover-based procedures often fail to predict system failure modes. Such inability of simplified procedures causes considerable error in predicted response. In order to solve this issue, an envelope-based pushover analysis procedure has been proposed, which assumes that each seismic response parameter is controlled by a predominant system failure mode, which may vary with respect to ground motion, its intensity and engineering demand parameter at a certain location in a structure. In the procedure it is assumed that the seismic demand associated with the first-mode pushover analysis is obtained by corresponding modal-based single-degree-of-freedom model, while demands for 'higher' failure modes are based on so-called failure-based single-degree-of freedom models. The total seismic demand is determined simply by enveloping the results based on several pushover analyses. Such approach enables approximate simulation of predominant system failure modes as function of ground motion and its intensity. By means of two reinforced concrete frame buildings and different sets of ground motions it is shown that the envelope-based pushover analysis procedure provides seismic demand for a given ground motion with a useful degree of accuracy. However, pushover analysis for invariant horizontal force distribution may not be always appropriate for predicting the most relevant system failure modes.

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Randomness of ground motions causes variability of system failure modes of a structure. Other parameters, which affect the formation of the failure modes, are the ground-motion intensity and sensitivity of the structure to variation of system failure mode. All these factors make it difficult to anticipate a system failure mode for a given ground motion without performing nonlinear response history analysis. Consequently, pushover-based procedures often fail to predict system failure modes. Such inability of simplified procedures causes considerable error in predicted response. In order to solve this issue, an envelope-based pushover analysis procedure has been proposed, which assumes that each seismic response parameter is controlled by a predominant system failure mode, which may vary with respect to ground motion, its intensity and engineering demand parameter at a certain location in a structure. In the procedure it is assumed that the seismic demand associated with the first-mode pushover analysis is obtained by corresponding modal-based single-degree-of-freedom model, while demands for 'higher' failure modes are based on so-called failure-based single-degree-of freedom models. The total seismic demand is determined simply by enveloping the results based on several pushover analyses. Such approach enables approximate simulation of predominant system failure modes as function of ground motion and its intensity. By means of two reinforced concrete frame buildings and different sets of ground motions it is shown that the envelope-based pushover analysis procedure provides seismic demand for a given ground motion with a useful degree of accuracy. However, pushover analysis for invariant horizontal force distribution may not be always appropriate for predicting the most relevant system failure modes.

Introduction

Accurate estimation of seismic demand of structures under strong ground motions is a complex task. Consequently, many simplified nonlinear procedures based on pushover analysis have been developed for simulating seismic response of structures. Basic pushover-based procedures (e.g. [1]) combine pushover analysis for an invariant distribution of lateral forces with response spectrum or history analysis of equivalent single-degree-of-freedom (SDOF) model. Such procedures are subject of limitations. Several procedures were developed to overcome the basic assumption that the structures vibrate predominantly in a single mode. For example, the invariant force vector can be replaced by adaptive distribution of horizontal forces [2, 3]. The higher mode

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effects in elevation can be considered by enveloping results of basic invariant pushover-based procedure and response spectrum analysis [4]. On the other hand, some procedures involve several pushover analyses. Chopra and Goel [5] developed modal pushover analysis (MPA) procedure, which considers the effects of the first few vibration modes in the analyzed direction of a structure by using the corresponding invariant distribution of horizontal forces for each analysis. The total seismic demand is determined by using appropriate combination of results. The basic MPA was a subject of different modifications and extensions, e.g. [6]. Recently, Sucuoğlu and Günay [7] proposed generalized force vectors for multi-mode pushover analysis, where total seismic demand is determined by enveloping the results of different pushover analyses.

Few studies addressed the question of the ability of the pushover-based procedures to predict seismic response of a structure due to a single ground motion. Bobadilla and Chopra [8] observed that the first-mode SDOF model can estimate the median roof displacement and maximum storey drifts of reinforced concrete building to a useful degree of accuracy, whereas this is not true for some ground motions. Brozovič and Dolšek [9] realized that the accuracy of simplified nonlinear procedures is affected by adequacy of pushover analysis to predict system failure modes, which is similar to that from nonlinear response history analysis (NRHA). According to this knowledge, an envelope-based pushover analysis (EPA) procedure [9] was proposed. It enables consideration of three system failure modes, which are simulated by using modal-based pushover analyses and by introducing so-called failure-based SDOF models. It has been shown that EPA procedure is sufficiently accurate even for estimation of seismic collapse risk of buildings [10].

In this paper, EPA procedure is summarized and used for simulation of system failure modes for a given ground motion. A discussion is given in order to provide an answer why pushover-based procedures usually fail to predict seismic response of buildings due to some specific ground motions, which, according to results of response history analysis, cause system failure modes that are significantly different to that observed from first-mode pushover analysis.

Summary of the envelope-based pushover analysis (EPA) procedure

The EPA procedure assumes that the seismic demand for each response parameter is controlled by a predominant system failure mode that may vary according to the ground motion and its intensity. In order to be able to simulate the most important system failure modes, several pushover analyses need to be performed, by analogy to MPA procedure [5], whereas the total seismic demand is determined by enveloping the results associated with each pushover analysis. The demand for the most common system failure mode resulting from the first-mode pushover analysis is obtained by response history analysis for the equivalent modal-based SDOF model, whereas demand for other failure modes is based on so-called failure-based SDOF models, which are described hereinafter. This makes the EPA procedure fully nonlinear and equivalent to the N2 procedure [1] provided that it involves only first-mode pushover analysis and response history analysis of the corresponding modal-based SDOF model.

Pushover analyses

Three pushover analyses are performed by using an invariant horizontal load distribution $\mathbf{s}_i$, which corresponds to the product of the diagonal mass matrix $\mathbf{M}$ and the ith vibration mode ϕ_i (i = 1, 2, 3) of the structure in the analyzed direction

$$\mathbf{s}_i = \mathbf{M} \cdot \phi_i \tag{1}$$

The global results of the pushover analyses are the three pushover curves, which are idealized in order to define the force-displacement relationships of the equivalent SDOF models. Storey drifts and the estimated level of damage to individual structural elements can be directly assessed from results of each pushover analysis at a given target roof displacement, which is determined by nonlinear dynamic analysis of an appropriate SDOF model.

Modal-based and failure-based SDOF models

The EPA procedure involves two types of SDOF models. In addition to modal-based SDOF models (e.g. [1, 5]), so-called failure-based SDOF models [9] are defined. The latter are intended to estimate the roof displacement in the case if ground motions cause significantly different system failure modes as those observed for the first-mode pushover analysis.

The force-displacement relationships of the 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. This factor is, by analogy to the MPA [5], 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. $\Gamma_{m,i}$ is in fact the modal participation factor for the ith natural vibration mode. The period of the ith modal-based SDOF model $T^*_{m,i}$ is defined as follows

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

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

Definition of the failure-based SDOF models takes into account so-called failure-mode displacement vector obtained from the pushover analysis instead of the mode shape. The transformation factors $\Gamma_{f,i}$ and equivalent SDOF masses $m^*_{f,i}$ of the failure-based SDOF models are therefore defined as follows

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

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

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

can differ slightly in comparison to $T^*_{m,i}$, if a difference between $m^*_{f,i}$ and $m^*_{m,i}$ occurs. It is assumed that the failure-mode displacement vector is associated with 80% strength in the softening branch of the ith-mode pushover curve [9]. 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 ($\Gamma_{f,1} \approx \Gamma_{m,1}$).

The ability of the equivalent SDOF models to predict the roof displacement also depends on their hysteretic and damping model. In this paper the peak-oriented model was used for response history analysis of the SDOF model [11]. It should be noted that EPA procedure assumes the same damping ratio for all three SDOF models. This is not consistent with the modal response history analysis [12]. This issue is discussed elsewhere [9].

Prediction of seismic demand

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

1) Perform pushover analyses for first, second and third mode invariant 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 target roof displacements with the modal-based and each of the failure-based SDOF model by using the nonlinear response history analysis.
4) Obtain the total seismic demand for each response parameter by enveloping the results associated with each pushover analysis at corresponding target roof displacements.

Simulation of system failure modes for two reinforced concrete frame buildings

Description of the structures and mathematical models

The system failure modes are simulated for the 8-storey (B8) and 15-storey building (B15) (Fig. 1), for which the influence of ground motions on the variation of system failure modes is significant. Buildings were designed for ground acceleration of 0.25 g, medium ductility class, behavior factor of 3.9 and soil types B and C, respectively, for 8-storey and 15-storey building.

Eurocode 8 [13] provisions were applied. In the case of 8-storey building, which is regular in elevation, the cross section dimensions of the columns and beams amounted to 50/50 cm, while the slabs have a thickness of 20 cm. In the case of 15-storey building, the dimensions of the cross section of the columns vary with the height of the building (70/80, 60/70 and 50/60 cm, respectively, for the 1st to 9th storey, the 10th to 12th storey and the 13th to top storey). On the other hand all the beams have equal dimensions of 55/60 cm and slab thickness amounted to 22 cm.

Simplified nonlinear models were built by using PBEE toolbox [11] in conjunction with OpenSees [14]. Beam and column flexural behavior was modeled by one-component lumped plasticity elements, consisting of an elastic beam and two inelastic rotational hinges. The typical tri-linear moment-rotation relationship is shown in Fig 1. Cyclic behavior in the plastic hinges of beams and columns was simulated by using uniaxial hysteretic material [14]. The deterioration under cyclic deformations was implicitly accounted for by the moment-rotation envelopes of plastic hinges. P-Delta effects were considered for all analyses. Mass proportional damping ratio at the fundamental period of vibration (5%) was assumed for all response history analyses. Details are available elsewhere [9, 11].

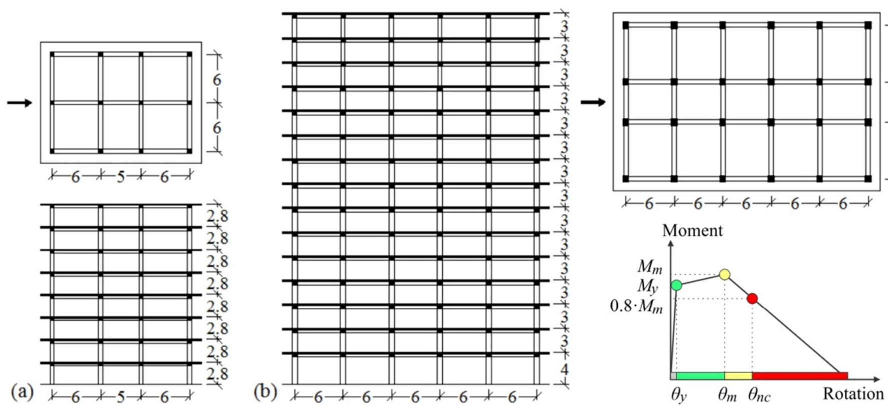

Figure 1. Elevation and plan views of (a) the 8-storey and (b) the 15-storey building, with the indicated direction of seismic loading. In addition, the envelope of the moment – rotation relationship in plastic hinges of columns and beams is shown.

Description of ground motion sets

Three sets of 40 ground motions were selected. Two of them are hazard-consistent since ground motions were selected [15] in order to match the conditional spectrum (CS) [16] defined for each of the two buildings and return period of 10000 years (Figs. 2a and 2b). The seismic hazard parameters needed for the definition of conditional spectra were determined by using EZ-FRISK [17] for site in Ljubljana, Slovenia. Note that conditioning periods were equal to fundamental periods T_1 of the buildings, which amounted to 1.23 s and 1.9 s, respectively, for 8-storey and 15-storey building. These two sets of ground motions are indicated as $A_{CS,B8}$ and $A_{CS,B15}$. The third set of ground motions (Fig. 2c), indicated as A_B, was selected by Baker et al. [18] in order to match the median and corresponding variance of elastic acceleration spectra defined by ground motion prediction model [19] for a magnitude 7 strike-slip earthquake at a distance of 10 km on soil site. Additional data about the ground motions from selected sets is available elsewhere [18, 20].

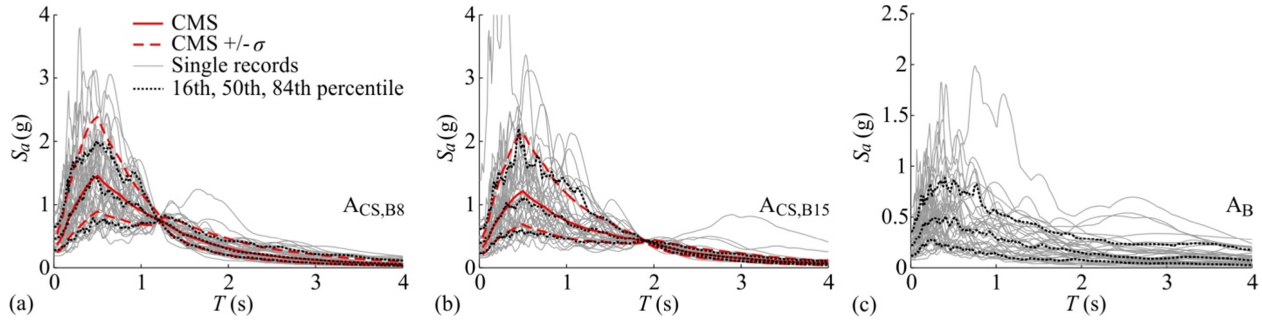

Figure 2. Single acceleration spectra of ground motions with corresponding 16th, 50th and 84th percentile for (a) set $A_{CS,B8}$, (b) set $A_{CS,B15}$ and (c) set A_B. In the case of hazard consistent sets also conditional mean spectrum and standard deviation is shown.

Pushover analyses and the SDOF models

Three pushover analyses were performed for one orthogonal direction of each building (Fig. 1). Distribution of horizontal forces was defined by Eq. 1 on the basis of the first three vibration modes. The maximum base shear versus weight ratio was approximately constant for all three pushovers and had a value of 0.11 and 0.07 for the 8-storey building and 15-storey building, respectively. The system failure modes obtained by pushover analyses at the near collapse limit state, which is assumed at 80% strength in the softening branch of the pushover curves, are presented in Figs. 3 and 4. Note that the grey, green, yellow and red colors represent damage in the plastic hinges (Fig. 1).

For each building one modal-based and two failure-based SDOF models, which are needed in the EPA procedure, were determined (Table 1). The periods T^* of modal-based SDOF models coincided with the natural vibration periods of the buildings, since the initial stiffness of the SDOF models was set equal to that obtained from the pushover analyses. Modal-based and failure-based SDOF models determined for the first-mode pushover analysis had similar characteristics, since fundamental mode shape is similar to corresponding deformation shape determined by pushover analysis from elastic range to near collapse limit state. In the case of second or third-mode pushover analyses, the difference between mode shape and deformation shape becomes significant. In spite of that, the values of periods T^* and equivalent masses m^* corresponding to the failure-based SDOF models differed in comparison to values, which could be defined for modal-based SDOF models for all pushover analyses on average only approximately 5% and 10%, respectively [9]. Significant differences were, however, observed between modal-based and failure-based transformation factors. Consequently yield accelerations S_{ay} of the failure-based SDOF models was observed from 2.5 to 6 times smaller in comparison to those based on modal-based SDOF models for second and third-mode pushover analysis [9]. It should be noted, that transformation factors Γ for all three SDOF models used in EPA are of similar order, approximately 1.25 (Table 1), 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 enable approximate simulation of the global seismic demand associated with failure modes that cannot be simulated by the modal-based SDOF models.

Table 1. The periods T^*, masses m^*, transformation factors Γ and yield accelerations of the capacity diagrams S_{ay} for the modal-based and failure-based SDOF models used in the EPA procedure for both buildings.

	8-storey building				15-storey building			
Mode (type)	T^* (s)	m^* (t)	Γ	S_{ay} (g)	T^* (s)	m^* (t)	Γ	S_{ay} (g)
1 (modal)	1.23	1471	1.28	0.13	1.90	7010	1.35	0.08
2 (failure)	0.41	575	1.23	0.33	0.72	2729	1.20	0.21
3 (failure)	0.25	446	1.20	0.45	0.39	1562	1.27	0.35

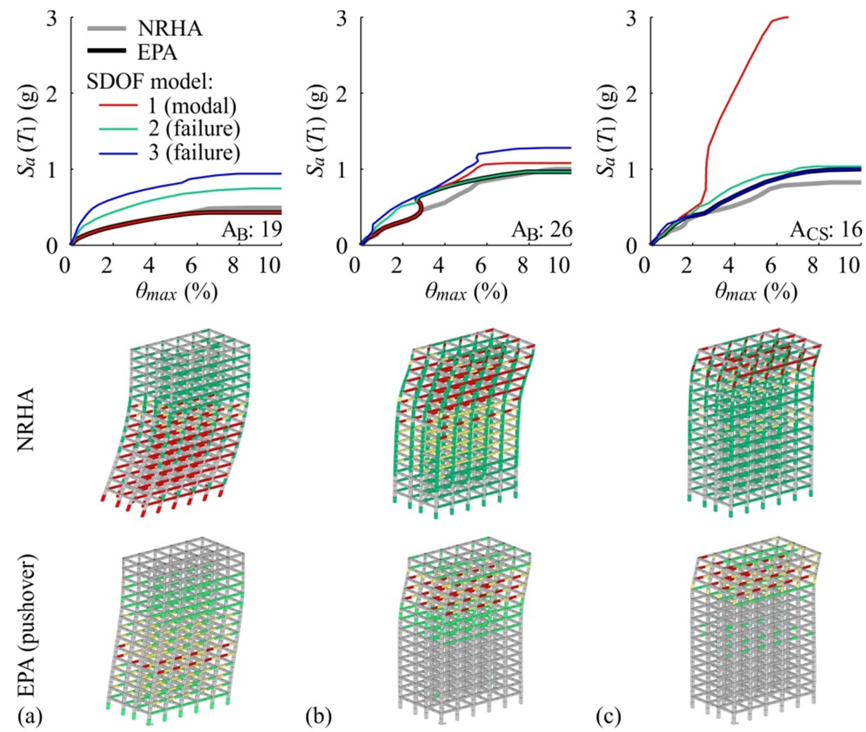

Figure 3. The IDA curves based on NRHA and EPA procedure for the 15-storey building and (a) 19th ground motion from set A_B, (b) 26th ground motion from set A_B and (c) 16th ground motion from set $A_{CS,B15}$. In addition, the approximate IDA curves associated with three SDOF models are presented together with the system failure modes, which cause collapse in the case of NRHA and EPA procedure.

Predominant system failure modes

Different system failure modes obtained by NRHA and EPA procedure, are presented in Fig. 3 for 15-storey building. It is interesting that some ground motions (e.g. 19th ground motion from set A_B, see Fig. 3a) cause formation of system failure mode, which can be appropriately simulated by first-mode pushover analysis. In this case the 1st modal-based SDOF model represents the predominant system failure mode, since the 'higher' failure-based SDOF models do not affect the seismic demand at any intensity level as shown by IDA curves (Fig. 3a). In such cases, basic pushover-based procedures can estimate seismic response of structures with sufficient accuracy. Some other ground motions cause formation of failure modes, which can be approximately simulated by second-mode (Fig. 3b) or third-mode pushover analysis (Fig. 3c). In these cases displacement demand of failure-based SDOF models is greater than that of the first modal-based SDOF model. Especially in the case of the 16th ground motion from set $A_{CS,B15}$ (Fig. 3c) it can be observed that the error in seismic demand obtained by using the first modal-based SDOF model, is extremely large. This error is practically eliminated in the case if seismic demand is obtained from failure-based SDOF models (Fig. 3c).

Similar results were obtained for the 8-storey building (Fig. 4). For 6th ground motion from set A_B, seismic demand can be accurately determined by simulating only the first system failure mode (Fig. 4a). For other failure modes (Fig. 4b and 4c), it is interesting to observe that results of NRHA show concentration of damage in the lower stories, whereas the EPA procedure simulates damage concentration at the top stories. In spite of this error, system failure mode develops in approximately equal number of stories by using NRHA or EPA, which determines appropriate global results in terms of maximum storey drift ratio. In general this limitation is the consequence of the invariant force-based pushover analysis, which is not capable to simulate all types of system failure modes which can be observed from NRHA. In such cases, EPA procedure provides conservative estimates of storey drifts in the upper part of the structure [9].

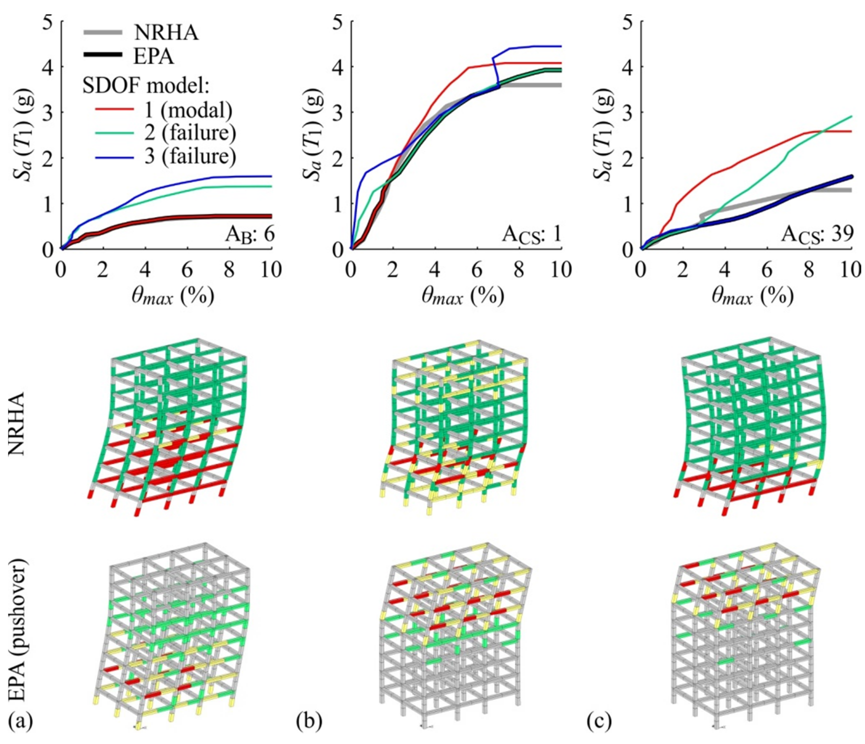

Figure 4. The IDA curves based on NRHA and EPA procedure for the 8-storey and (a) 6th ground motion from set A_B, (b) 1st ground motion from set $A_{CS,B8}$ and (c) 39th ground motion from set $A_{CS,B8}$. In addition, the approximate IDA curves associated with three SDOF models are presented together with the system failure modes, which cause collapse in the case of NRHA and EPA procedure.

Consideration of appropriate system failure modes in EPA procedure significantly improves simulation of demand parameters, which can be observed from Fig. 5, where EPA-based IDA curves are presented in different colors, indicating the predominant system failure mode, which controlled the maximum storey drift. As expected, taller building was more sensitive to 'higher' system failure modes. However, sensitivity to variation of system failure modes also significantly depended on the ground motion set. In the case of hazard-consistent sets of ground motions ($A_{CS,B8}$ and $A_{CS,B15}$), the 'higher' failure modes controlled the demand parameter, whereas for the ground-motion set A_B the first failure mode often caused greater demand. It can also be observed that controlling system failure mode depended on the ground motion intensity. The seismic demand at lower intensities was more often governed by the first failure mode, while in the case of greater intensities 'higher' modes often prevailed (Fig. 5). Additionally, the percentile IDA curves based on NRHA, EPA and basic pushover-based procedure (PA1) are presented. It is obvious that PA1 procedure provided sufficiently accurate results if first system failure mode governed seismic response for most ground motions (B8, A_B) However, this is not the case for the majority of examples presented in Fig. 5.

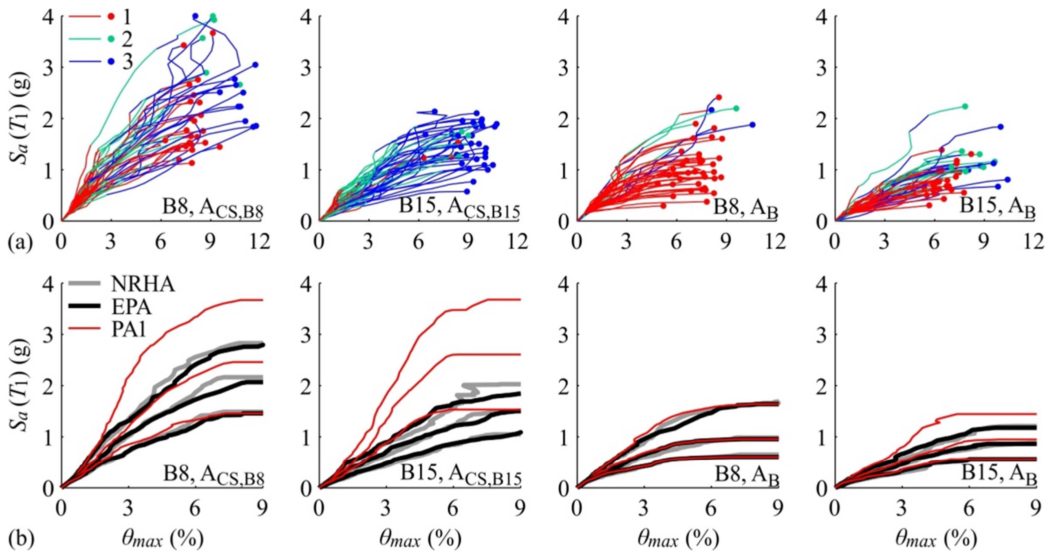

Figure 5. (a) EPA-based IDA curves with indicated predominant system failure modes obtained from first-mode (1), second-mode (2) and third-mode (3) pushover analysis and (b) 16^{th}, 50^{th} and 84^{th} percentile IDA curves based on NRHA, EPA and PA1 for both buildings (B8 and B15) and all sets of ground motions ($A_{CS,B8}$, $A_{CS,B15}$ and A_B).

Conclusions

Simulation of system failure modes is an important component of simplified nonlinear procedures for seismic performance assessment. If the structure is sensitive to formation of different failure modes, than pushover analysis procedure should account for variation of plastic mechanism due to the randomness of ground motion. In the paper it is shown that the EPA procedure, which involves failure-based SDOF models and is based on enveloping results for several system failure modes, can sufficiently solve this issue. However, pushover analysis based on invariant load distribution is not always capable to predict the most relevant system failure modes observed due to ground motions.

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