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SELECTION OF FEW GROUND MOTIONS FOR DECISION MAKING ASSOCIATED WITH NO-COLLAPSE REQUIREMENT

Marko BROZOVIČ[1] and Matjaž DOLŠEK[2]

ABSTRACT

The practice-oriented method, which can be used to check the reliability against collapse of a structure by few dynamic analysis of nonlinear structural model, is summarized. The method involves two steps for selection of few ground motions. In the first step, a set of hazard-consistent ground motions is selected from a representative database. The second step utilizes the results of a simplified pushover-based seismic demand analysis, which is used to assess the proxy for collapse intensities. Based on the approximate results, few (e.g. seven) characteristic ground motions are selected in the vicinity of the characteristic percentile (e.g. 16th percentile) of collapse intensity. The nonlinear dynamic analyses at the level of structure are then performed for characteristic ground motions only, which are scaled to the characteristic value of target collapse intensity associated with the target collapse risk. Such approach enables straightforward decision making regarding the no-collapse requirement. It is decided that the collapse risk is less than the target collapse risk if the collapse is attained for less than half of the characteristic ground motions. It is shown that the decision about the acceptability of the structure with regard to the no-collapse requirement is made with sufficient degree of accuracy, which is demonstrated by means of 15-storey reinforced concrete building. The described method for design check based on nonlinear dynamic analysis may be attractive for implementation in building codes.

INTRODUCTION

Decisions about adequate design of structures in seismic regions should be based on assessment of seismic risk, which can be expressed in the simplest case by mean annual frequency of exceeding a designated limit state. This is challenging and ambitious goal, which can be realized if the concept of design of structures in building codes will be modernized. However, comprehensive design procedures, which enable design of structures for target collapse risk (e.g. Lazar and Dolšek, 2013; Dolšek, 2013), already exist. Such design procedures explicitly takes into account risk aversion associated with the consequences of collapse or damage of the structure. In general, the target seismic risk depends on potential fatalities, direct and indirect economic losses, which can be caused by earthquakes in the prescribed lifetime of the structure. However, the target seismic risk is to some extent subjective performance measure. Consequently, a maximum value of acceptable seismic risk should be defined by the authorities in order to achieve an adequate degree of societal resilience, whereas an owner should have a possibility to live and work in a facility for which the estimated seismic risk is even below the maximum acceptable seismic risk. Nevertheless, all buildings in seismic regions should be adequately designed for no-collapse requirement, since the most important objective in earthquake engineering is prevention of loss of life.

[1] Assistant, Faculty of Civil and Geodetic Engineering, Ljubljana, Slovenia, marko.brozovic@fgg.uni-lj.si
[2] Associate Professor, Faculty of Civil and Geodetic Engineering, Ljubljana, Slovenia, mdolsek@fgg.uni-lj.si

In order to check the design, the collapse risk should be estimated and compared to the target collapse risk. In general, the collapse risk is estimated by integrating the risk equation which combines seismic hazard with collapse fragility function. Many procedures based on nonlinear dynamic analysis are available to assess the collapse fragility function. However, the number of simulations is often large, which disables to use the dynamic analysis of nonlinear structural model for practical applications. Less computationally demanding procedures are therefore needed. Eads et al. (2013) proposed an efficient method for estimating collapse risk on basis of the dynamic analysis at two levels of intensities. In this case the collapse fragility function is estimated using results of dynamic analysis at two intensities and assumption of lognormal distribution of collapse intensities. However, this procedure still requires a large set of ground motions, which makes the procedure computationally demanding.

In this paper a selected variant of the 3R method (Dolšek and Brozovič, 2014), which is used for design check on the basis of few dynamic analyses, is summarized. Few ground motions, which are selected in two steps, are scaled to co-called characteristic value of target collapse intensity associated with the target collapse risk. In the first step any procedure for selection of the hazard-consistent set of ground motions can be used, whereas the second step involve simplified pushover-based method in order to assess the proxy for collapse intensities. Firstly, a step by step explanation of a variant of the 3R method is given. For simplicity, the assessment of the sufficiency of the method for a range of structures is omitted in this paper since it is addressed elsewhere (Dolšek and Brozovič, 2014). In the second part of the paper, the method is demonstrated by means of an example of 15-storey reinforced concrete frame building.

SUMMARY OF THE VARIANT OF THE 3R METHOD FOR CHECKING THE ADEQUACY OF COLLAPSE RISK OF A STRUCTURE ON THE BASIS OF FEW NONLINEAR DYNAMIC ANALYSES

The 3R method is based on the assumption that the adequacy of collapse risk λ can be determined with intensity-based assessment at a single value of so-called target collapse intensity (Dolšek and Brozovič, 2014). Such an approach requires assumption regarding the distribution of the collapse fragility function. In the case if collapse fragility function is assumed lognormal then it is necessary to predefine the standard deviation of collapse intensities in log domain β_t in order to assess the target collapse intensity. Dolšek and Brozovič (2014) have showed that the target collapse intensity used in intensity-based assessment should be associated with so-called characteristic value of target collapse intensity $S_{a,ct}$ which corresponds to a low percentile from target collapse fragility function, which is called a characteristic percentile. The adequacy of the collapse risk of a structure can then be checked by performing dynamic analyses of nonlinear structural model for a set of hazard-consistent ground motions which are scaled to the $S_{a,ct}$. If the fraction of ground motions that cause the collapse of structure is lower than the characteristic percentile, it can be concluded that the structure is safe against collapse due to earthquakes. In the opposite case, the performance objective is not met.

It should be noted that the required number of dynamic analyses from the described procedure would be equal to the number of ground motions from a hazard-consistent set, which could still be large. Additional assumption is therefore needed in order to reduce the number of required dynamic analyses. It is assumed that a suitable characteristic ground motions (CGMs) can be selected from the initial hazard-consistent set using proxy for collapse intensities, which can be obtained by the pushover-based methods.

A selection of the CGMs, which is a subset of hazard-consistent set of ground motions, represents a second step of the ground motion selection procedure. The selected subset of ground motions corresponds to proxy for collapse intensities, which are close to the proxy for characteristic value of collapse intensity $S_{a,cp}$ associated with the characteristic percentile. It is also required that the proxy for median value of collapse intensities corresponding to CGMs $\tilde{S}_{a,p,cgm}$ closely match with the $S_{a,cp}$. The adequacy of the collapse risk of a structure can then be checked by performing the

nonlinear dynamic analysis for CGMs scaled to $S_{a,ct}$. If less than 50% of CGMs cause the collapse of a structure, it can be concluded that the performance objective was met and vice versa.

It should be noted that spectral acceleration at fundamental period of structure $S_a(T_1)$ is used as intensity measure in this paper. In addition, it is assumed that the number of the CGMs is equal to seven. In this case the initial set must contain at least 19 ground motions. If there are less than 19 ground motions, $\tilde{S}_{a,p,cgm}$ would always be greater than the $S_{a,cp}$, which would reduce the accuracy of the method. However, the hazard-consistent set can contain a large number of ground motions, since the proxy for collapse intensity is obtained by selected simplified method of analysis, which is not computationally demanding. Furthermore, 16th percentile is used as characteristic percentile. There are several reasons for such a decision. For example, (i) the value of the assumed standard deviation of collapse intensities in log domain β_t has small impact on characteristic value of target collapse intensity if it is defined in the vicinity of 16th percentile (Dolšek and Brozovič, 2014), (ii) scale factors of ground motions are more likely in the range which still allow unbiased estimates of seismic demand, (iii) the accuracy of simplified methods to provide the proxy for collapse intensities is greater for those ground motions which cause the collapse of buildings at low intensities (Brozovič and Dolšek, 2014) and (iv) it is well known that the intensities which have the largest contribution to the collapse risk occur in the lower half of the collapse fragility function (e.g. Eads et al., 2013).

The method for checking the adequacy of the collapse risk of a structure on the basis of few nonlinear dynamic analyses involves the following steps:

1) Perform probabilistic seismic hazard analysis at a site of interest and select a set of hazard-consistent ground motions. Several methods for selection of ground motions can be used and the set of ground motions from this step can be large.
2) Determine proxy for collapse intensities for all ground motions from a hazard-consistent set and assess the proxy for standard deviation of natural logarithms of collapse intensities β_p. Different simplified methods can be used to obtain the proxy for the collapse intensities.
3) Determine the characteristic value of target collapse intensity $S_{a,ct}$ from the target collapse fragility function, which is associated with the target collapse risk λ_t. In the simplest case the risk equation in closed-form can be used, as described below (see Eq.(3)). In this case, hazard function from step 1 has to be approximated by the analytical seismic hazard function, whereas the standard deviation of the collapse intensities can be assumed from the step 2.
4) Define a subset of 7 CGMs on basis of the proxy for collapse intensities obtained in step 2. The proxy for median value of collapse intensities for CGMs $\tilde{S}_{a,p,cgm}$ should be as close as possible to the proxy for characteristic value of collapse intensity $S_{a,cp}$. In addition, the standard deviation of collapse intensities corresponding to CGMs should be minimized.
5) Perform dynamic analysis for all 7 CGMs using a nonlinear model of entire structure. On the basis of collapse ratio r_C, which is defined as the ratio between the number of CGMs that caused collapse and the number of all CGMs, estimate whether the collapse risk of a structure is acceptable ($r_C < 0.5$) or not ($r_C > 0.5$).

The first step of the method requires probabilistic seismic hazard analysis, which has to be performed at the site of a structure. A larger set of ground motions can be selected in accordance with results of seismic hazard analysis in order to obtain a sample of ground motions which is consistent with the hazard. Different procedures can be used for this purpose. For example, the conditional spectrum proposed by Baker (2011) can be prescribed as a target spectrum, whereas the set of ground motions can be obtained by computationally efficient ground motion selection algorithm proposed by Jayaram et al. (2011). This procedure is used later on in the example. However, there is no need that the ground motions are selected to match the target spectrum.

In the second step, the proxy for collapse intensities has to be determined. A wide range of simplified methods exists. Any method which can provide an approximate value (proxy) of the collapse intensity for a given ground motion can be used. Two simplified methods will be used later in the example. The PA1 procedure involves the first-mode pushover analysis and the dynamic analysis of single-degree-of-freedom (SDOF) model. This procedure is equal to N2 method (Fajfar, 2000) provided that the dynamic analysis for SDOF model replaces inelastic response spectra. As an

alternative to this conventional method, an envelope-based pushover analysis (EPA) procedure (Brozovič and Dolšek, 2014) will also be used. It provides results with a useful degree of accuracy also for taller buildings, where collapse risk is significantly affected by more than one system failure mode. The result of the used simplified method is the proxy for collapse fragility function with lognormal distribution. Therefore the proxy for collapse fragility function can be described by two parameters, i.e. the proxy for median value of collapse intensity $\tilde{S}_{a,p}$ and corresponding standard deviation of natural logarithms β_p, which can be obtained from sample of proxy for collapse intensities estimated for all ground motions from a hazard-consistent set. For example, the parameters of the proxy for collapse fragility function can be estimated according to the method of moments

$$\tilde{S}_{a,p} = \overline{S}_{a,p} \cdot e^{-0.5\beta_p^2} \tag{1}$$

$$\beta_p = \sqrt{\ln\left(\frac{\sigma_p{}^2}{\overline{S}_{a,p}^2} + 1\right)} \tag{2}$$

where $\overline{S}_{a,p}$ and σ_p are, respectively, the mean and corresponding standard deviation of sample values of proxy for collapse intensities.

In the third step, the characteristic value of target collapse intensity $S_{a,ct}$ is derived from the target collapse risk λ_t. If the value of λ_t, which is considered as acceptable collapse risk, is not assessed by the risk aversion of the regional society, it can be obtained utilizing models of acceptable collapse risk, e.g. Helm (1996) or CEN (2004a). It should be noted, that different models provide different results, while there is no generally accepted model yet (Lazar and Dolšek, 2012). Furthermore, such models often provide acceptable probability of collapse, which can be assumed equal to mean annual frequency of collapse, i.e. collapse risk, since these values are usually very low. Once the target collapse risk is selected, the median value of target collapse intensity $\tilde{S}_{a,t}$ can be determined utilizing risk equation in closed-form or iteratively by numerical integration of the risk equation. In both cases, the standard deviation of collapse intensities in log domain β_t has to be assumed or adopted from the estimated value in step 2 ($\beta_t = \beta_p$). If the closed-form solution of the risk equation is used, the median value of target target collapse intensity $\tilde{S}_{a,t}$ can be determined from the following equation (Cornell, 1996)

$$\lambda_t = H(\tilde{S}_{a,t}) \cdot e^{0.5 \cdot k^2 \cdot \beta_t^2} \tag{3}$$

where $H(\tilde{S}_{a,t})$ is mean annual frequency of exceeding $\tilde{S}_{a,t}$ obtained from hazard curve and k represent the slope of linear approximation to the hazard curve in log domain. However, the characteristic value of target collapse intensity $S_{a,ct}$ is then estimated at 16th percentile of the target collapse fragility function as follows

$$S_{a,ct} = \tilde{S}_{a,t} \cdot e^{-\beta_t} \tag{4}$$

A subset of 7 CGMs is selected in step 4 taking into account the proxy for collapse intensities from the step 2. These ground motions cause collapse in the vicinity of proxy for characteristic value of collapse intensity $S_{a,cp}$. Note that the method for design check is sufficiently accurate if the accuracy of the pushover-based method for the estimation of the proxy for collapse intensities is sufficient. This issue is further addressed elsewhere (Dolšek and Brozovič, 2014).

Finally, the dynamic analyses are performed for CGMs, which are scaled to $S_{a,ct}$. Based on the results of dynamic analyses the decision regarding the collapse safety is straightforward. If collapse of a structure is observed for less than 50% of analyses (collapse ratio $r_C < 0.5$), the collapse safety is adequate since the collapse risk λ is assumed to be lower than the target collapse risk λ_t (Fig.1a). On the other hand, if the collapse risk λ is estimated to be beyond target collapse risk λ_t, i.e. $r_C > 0.5$ (Fig.1b), the results of dynamic analyses at the structural level can be used to indicate the weakest parts of the building in order to increase the strength and the ductility of the critical structural

components. The described practice-oriented method cannot be used to assess the value of collapse risk λ. However, the value of collapse ratio r_C indicates the difference between the λ and the λ_t.

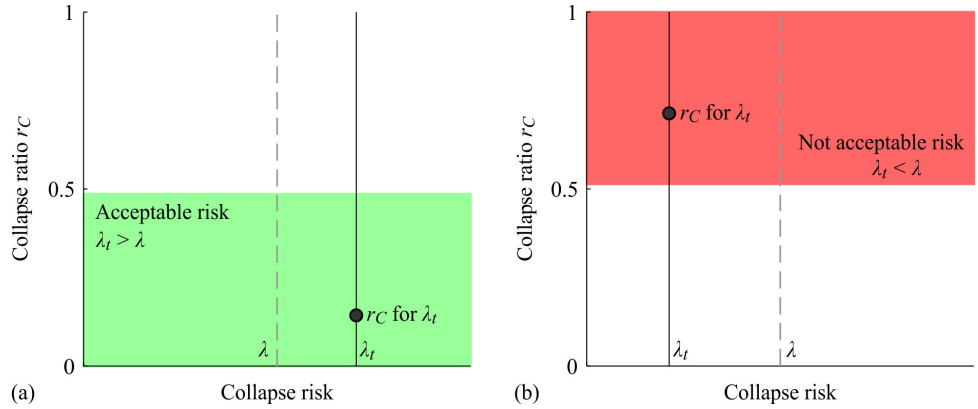

Figure 1. The decision making regarding the no-collapse requirement in the case if (a) collapse ratio $r_C < 0.5$ or (b) $r_C > 0.5$.

CASE STUDY FOR THE 15-STOREY FRAME BUILDING

The method for decision making regarding the no-collapse requirement is demonstrated by means of an example of a 15-storey reinforced concrete frame building. After description of the test structure, the method is presented in 5 steps described in previous section. Note that a proxy for collapse intensities (step 2) was obtained on the basis of two simplified seismic demand analysis procedures (PA1 and EPA procedure). In addition, the dynamic analysis for a nonlinear model of entire structure was used in order to get the point of comparison.

Description of the test structure and the model

The 15-storey reinforced concrete frame building, which is sensitive to system-failure-mode effects due to ground motions (Brozovič and Dolšek, 2014), was selected as a test structure. It was designed as earthquake-resistant structure in compliance with Eurocode 8 (CEN, 2004a) taking into account the design peak ground acceleration of 0.30 g, soil class B, ductility class medium and behaviour factor of 3.9. Elevation and plan view of the building is shown in Fig.2. 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. More details regarding the building are given elsewhere (Brozovič and Dolšek, 2014).

A simplified nonlinear model was built using PBEE toolbox (Dolšek, 2010) in conjunction with OpenSees (2011). The masses and mass moments of inertia were concentrated in the centre of gravity of floor diaphragms, which were assumed to be rigid in their own planes. Effective width of the beams is modelled according to Eurocode 2 (CEN, 2004b), 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. The moment-rotation relationship was determined according to PBEE toolbox (Dolšek, 2010). Cyclic behaviour in the plastic hinges of beams and columns was simulated by uniaxial hysteretic material available in OpenSees (2011). Unloading stiffness was defined by parameter β_U, which was 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).

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

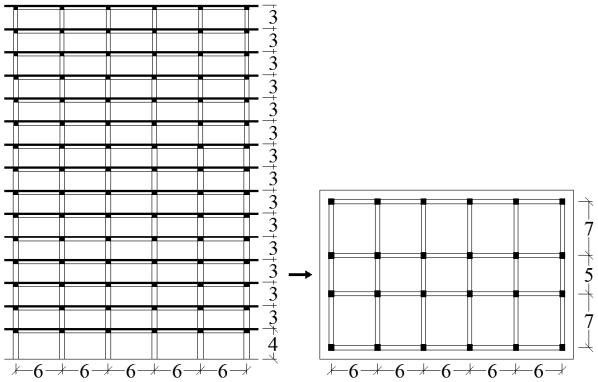

Figure 2. The elevation and plan view of the 15-storey building with indicated direction of seismic loading.

Hazard-consistent ground motion selection (step 1)

In general, ground motions used for seismic performance assessment should comply with seismic hazard at the site of the structure. In order to obtain a hazard-consistent set of ground motions conditional spectrum (Baker, 2011) was used as target spectrum. It was defined for conditioning period, which was set equal to the fundamental period of 15-storey building, T_1 = 1.9 s and 10.000 year return period. The parameters of seismic hazard, which are required for the definition of conditional spectrum, were determined by probabilistic seismic hazard analysis, which was performed for central Slovenia, city of Ljubljana. For this purpose EZ-FRISK (2012) was used, which involve simplified model of seismic sources of central Europe.

Furthermore, Sabetta and Pugliese (1996) ground motion prediction model and correlation model proposed by Baker and Jayaram (2008) was employed to obtain the target conditional spectrum. A set of 40 ground motions was then selected by computationally efficient ground motion selection algorithm for matching a target response spectrum mean and variance (Jayaram et al., 2011). Ground motions were selected only on the basis of matching the scaled elastic acceleration spectra with the target conditional spectrum (Fig.3), while other criteria (e.g. magnitude range, fault-to-site distance, soil type) were not considered. Note that the scale factors of ground motions were limited to be smaller or equal to 4. Additional information about the selected ground motions are available elsewhere (Brozovič, 2013).

Proxy for collapse intensities and corresponding distribution (step 2)

Proxy for collapse intensities was estimated by simplified seismic demand analysis procedures, which are not computationally demanding. Firstly, PA1 procedure was used, which is based on conventional pushover analysis. Additionally, envelope-based pushover analysis (EPA) procedure (Brozovič and Dolšek, 2014) was employed, since the 15-storey building is sensitive to the formation of different system failure modes. The EPA procedure determines the seismic demand as an envelope of results obtained by three simplified models, which are used to simulate three different system failure modes. The results based on the simplified procedures for seismic performance assessment were also checked by the 'exact' dynamic analysis of nonlinear model of entire structure, which provided a point of comparison. The results of seismic performance assessment of the structure using the three procedures are presented in Fig.4 in terms of IDA curves (Vamvatsikos and Cornell, 2002), where spectral acceleration at fundamental period of the building $S_a(T_1)$ and maximum inter-storey drift ratio θ_{max} is used for intensity measure and engineering demand parameter, respectively.

Parameters of the lognormal distribution of proxy for collapse intensities were calculated by method of moments. Sample mean $\bar{S}_{a,p}$ and standard deviation σ_p were firstly determined based on all three analysis procedures (Table.1). The corresponding proxy for median value of collapse intensity $\tilde{S}_{a,p}$ and standard deviation of natural logarithms β_p were then obtained by Eq.(1) and Eq.(2), respectively (Table.1). PA1 procedure provided approximately 60% higher mean and median values of proxy for collapse intensities in comparison to the point of comparison based on dynamic analysis. This error was reduced with EPA procedure to 6% and 3% in the case of sample mean $\bar{S}_{a,p}$ and estimated median $\tilde{S}_{a,p}$, respectively. However, the standard deviation of natural logarithms β_p obtained by PA1 procedure was quite accurately estimated (9% error), while EPA procedure determined the β_p with around 30% error in comparison to that based on dynamic analysis (Table.1).

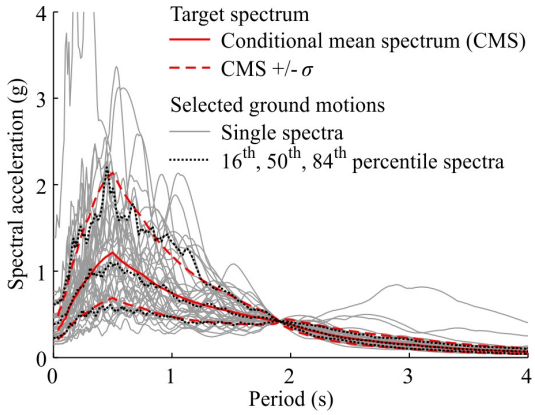

Figure 3. Response spectra of 40 hazard-consistent ground motions and corresponding 16th, 50th and 84th percentiles compared to the target conditional spectrum defined for a conditioning period equal to T_1 = 1.9 s, 10.000 year return period and location in central Slovenia, city of Ljubljana.

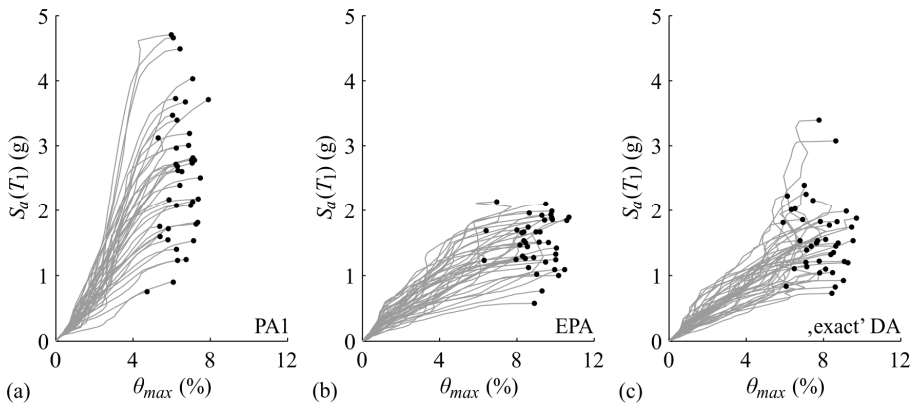

Figure 4. The IDA curves determined by (a) PA1, (b) EPA and (c) 'exact' dynamic analysis procedure and the corresponding collapse intensities for 15-storey building and hazard-consistent set of 40 ground motions.

Table 1. The sample mean and corresponding standard deviation of proxy for collapse intensities ($\bar{S}_{a,p}$ and σ_p) based on PA1, EPA and 'exact' dynamic analysis (DA) procedures (see Fig.4) with the corresponding parameters of lognormal distribution ($\tilde{S}_{a,p}$ in β_p).

	PA1	EPA	'exact' DA
$\bar{S}_{a,p}$ (g)	2.57	1.51	1.60
σ_p (g)	1.00	0.37	0.57
$\tilde{S}_{a,p}$ (g)	2.39	1.46	1.50
β_p (g)	0.38	0.24	0.35

Target collapse intensity associated with the target collapse risk (step 3)

In this study the target collapse risk λ_t was set to $5 \cdot 10^{-6}$. The corresponding median value of target collapse intensity $\tilde{S}_{a,t}$ can be determined by Eq.(3) for a known seismic hazard curve and the value of standard deviation β_t, which is assumed equal to β_p (Table.1). Note that the $\tilde{S}_{a,t}$ was calculated with iterative procedure, since the parameter k (see Eq.(3)) depends on the value of $\tilde{S}_{a,t}$. In each iteration the parameter k was calculated on the basis of $\tilde{S}_{a,t}$ from the previous step, till the differences in $\tilde{S}_{a,t}$ were negligible. Then the characteristic value of target collapse intensity $S_{a,ct}$ was determined by Eq.(4). In Table.2 it can be observed, that values of $S_{a,ct}$ are obtained with sufficient degree of accuracy taking into account the β_p obtained by PA1 or EPA procedure for estimation of β_t. Consequently the β_t is not very significant parameter when characteristic value of target collapse intensity corresponds to the 16th percentile (Dolšek and Brozovič, 2014).

Table 2. The median value of target collapse intensity $\tilde{S}_{a,t}$, the characteristic value of target collapse intensity $S_{a,ct}$ and the proxy for characteristic value of collapse intensity $S_{a,cp}$ determined on basis of three seismic performance assessment procedures.

	PA1	EPA	'exact' DA
$\tilde{S}_{a,t}$ (g)	1.88	1.71	1.83
$S_{a,ct}$ (g)	1.29	1.34	1.30
$S_{a,cp}$ (g)	1.54	1.12	1.10

Selection of characteristic ground motions (step 4)

Before dynamic analysis is performed for entire structural model at $S_{a,ct}$, a subset of ground motions is selected in order to additionally reduce the computational time. From the initial hazard-consistent set of 40 ground motions a subset of only 7 characteristic ground motions (CGMs) was selected on the basis of proxy for collapse intensities from step 2. The 7 CGMs were selected in the vicinity of the proxy for characteristic value of collapse intensity $S_{a,cp}$ in such a way, that the proxy for median value of collapse intensity corresponding to CGMs $\tilde{S}_{a,p,cgm}$ matched with the $S_{a,cp}$ (Fig.5). Note that the value of $S_{a,cp}$, which was in this case estimated by PA1, EPA and for comparison reasons also by 'exact' dynamic analysis procedure, is generally different in comparison to the characteristic value of target collapse intensity $S_{a,ct}$, which corresponds to the target collapse risk λ_t (Table.2). It can be observed that the $S_{a,cp}$ estimated by PA1 procedure is approximately 40% higher than that based on EPA or 'exact' dynamic analysis procedure (Table.2 and Fig.5).

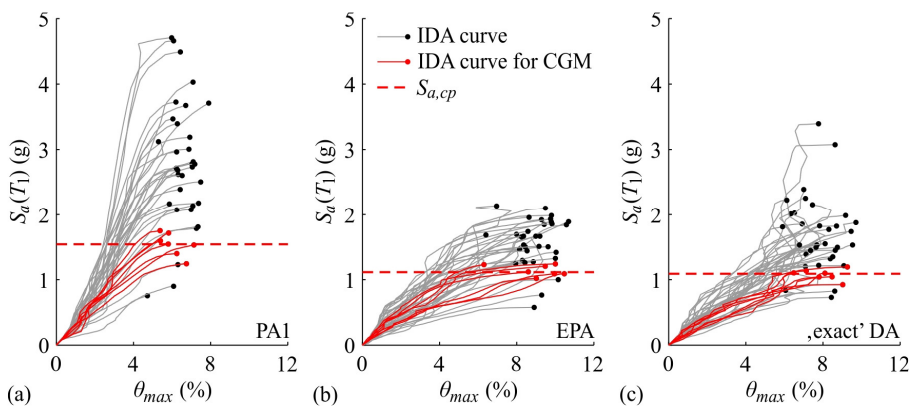

Figure 5. The IDA curves for the 15-storey building and hazard-consistent set of 40 ground motions with indicated IDA curves corresponding to 7 CGMs causing collapse in the vicinity of the proxy for characteristic value of collapse intensity $S_{a,cp}$ in the case of (a) PA1, (b) EPA and (c) 'exact' dynamic analysis procedure.

The accuracy of the PA1 and EPA procedure for the selection of CGMs is demonstrated in Fig.6 using the IDA curves obtained on the basis of dynamic analysis for nonlinear structural model. It can be observed that the median value of collapse intensities corresponding to CGMs based on simplified procedures (PA1 or EPA) is similar to the $S_{a,cp}$ determined by 'exact' dynamic analysis. Consequently it can be concluded that both simplified methods were sufficiently accurate to select appropriate CGMs, although $S_{a,cp}$ was not estimated with sufficient accuracy in the case of PA1 procedure.

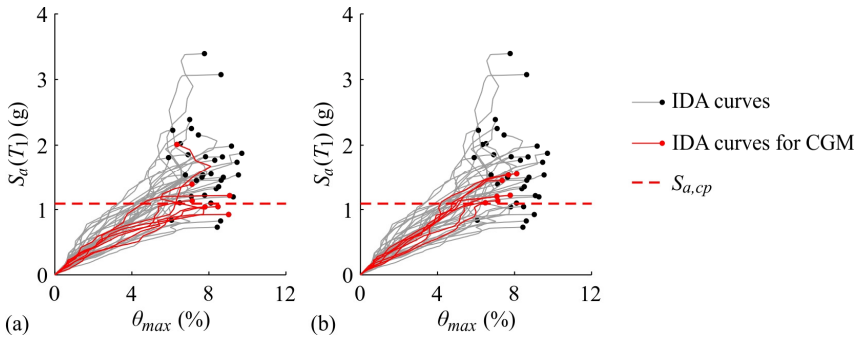

Figure 6. The IDA curves determined by 'exact' dynamic analysis for the 15-storey building and hazard-consistent set of 40 ground motions with indicated IDA curves corresponding to 7 CGMs selected on the basis of results obtained by (a) PA1 and (b) EPA.

Design check for no-collapse requirement on the basis of 7 characteristic ground motions (step 5)

The dynamic analyses were performed using nonlinear model of entire structure and the 7 CGMs, which were scaled to the characteristic value of target collapse intensity $S_{a,ct}$. On the basis of the obtained results collapse ratio r_C was determined. It amounted to 5/7, 4/7 and 7/7, respectively, in the case of using CGMs selected by PA1, EPA and 'exact' dynamic analysis procedure (Fig.7). More than 50% of CGMs caused collapse of the structure ($r_C > 0.5$) regardless of the method, which was used to select CGMs. Therefore, it is decided that the structure is not safe against collapse due to earthquakes. This means that the seismic collapse risk of the building is assumed as not acceptable, since the collapse risk λ is estimated to be greater than the target collapse risk ($\lambda_t = 5 \cdot 10^{-6}$). Therefore, the critical structural elements of the building should be strengthed and additional design check should be performed, until the performance objective is not met.

The accuracy of the practice-oriented method can be evaluated by direct assessment of the collapse risk λ on the basis of 'exact' dynamic analysis results, which is estimated to be $\lambda = 8 \cdot 10^{-6}$ (Fig.7). Since collapse risk is larger than the target collapse risk ($\lambda > \lambda_t$), it is shown that in the case of presented case study correct decision regarding the no-collapse requirement was made.

For comparison, collapse risk λ was estimated also by PA1 and EPA procedures. If the EPA procedure was used, the collapse risk λ was underestimated for only 8% ($7.3 \cdot 10^{-6}$) in comparison to the 'exact' result obtained by dynamic analysis. In the case of PA1 procedure quite large errors (66%) were observed ($2.7 \cdot 10^{-6}$) (Fig.7). However, if this error is expressed in terms of reliability index, it amounted to around 5% only, which is less than the difference between two reliability indexes prescribed in building codes (e.g. CEN, 2002). Furthermore, it was interesting that the PA1 procedure was sufficiently accurate for selection of CGMs, although PA1 was not sufficiently accurate for estimation of collapse risk. Obviously, the practice-oriented method for design check of no-collapse requirement is more reliable than direct estimation of seismic risk on basis of simplified analysis procedures. On the other hand, the computational time needed to assess the adequacy of the seismic collapse risk utilizing the described method is reduced to approximately 1% of the required computational time for estimation of collapse risk utilizing incremental dynamic analysis.

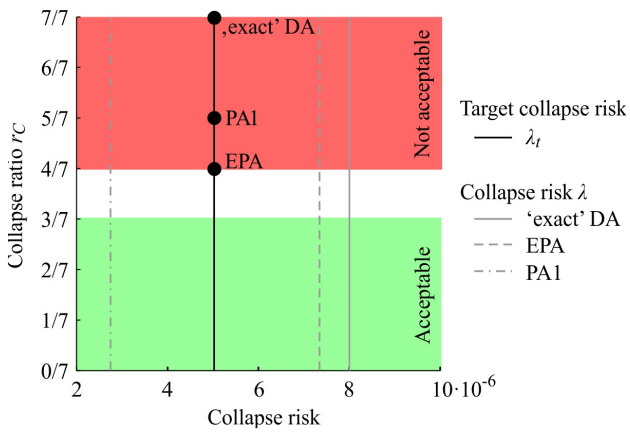

Figure 7. The decision making regarding the no-collapse requirement using the collapse ratio r_C corresponding to CGMs, which were selected according to PA1, EPA, and for comparison also on the basis of 'exact' dynamic analysis. In addition, the target collapse risk λ_t and the collapse risk λ, determined with the three seismic performance assessment procedures, are presented with vertical lines.

CONCLUSIONS

The variant of 3R method was presented and demonstrated by means of example of 15-storey reinforced concrete frame building. The method is intended to check the adequacy of collapse risk of structures on the basis of few nonlinear dynamic analyses. For this reason a two-step procedure for the selection of a few characteristic ground motions is used. It was shown that the simplified analysis procedure PA1, which uses conventional pushover analysis and the dynamic analysis of the equivalent SDOF model, was not sufficiently accurate for the assessment of collapse risk in the case of the 15-storey building. However, the characteristic ground motions selected by using results of the PA1 procedure were appropriate for checking the adequacy of collapse risk of the structure. Similar conclusion can be made for the characteristic ground motions obtained by the EPA procedure, which, however, provided estimate of collapse risk with acceptable accuracy.

The 3R method can be used to decide whether the collapse risk of investigated structure is larger or lower in comparison to the target collapse risk, which is assumed tolerable. It was shown that the computational time, which was needed to assess the adequacy of the collapse risk, was reduced to approximately 1% of the required computational time needed for assessment of collapse risk on basis of incremental dynamic analysis. The efficiency would be even greater if the hazard-consistent set would contain more ground motions. In addition to the reduction of the computational time, smaller scale factors of ground motions are required, which avoid biased estimates of seismic demand.

The practice-oriented method for decision making regarding the collapse risk acceptability can be applied to existing and new structures, or it can be used within an iterative risk-based design procedures based on dynamic analysis of structures. Note that the theoretical background and general description of the 3R method is given elsewhere (Dolšek and Brozovič, 2014).

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