Default Dispersion Measures for Simplified Probabilistic Assessment of RC Frames

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SUMMARY:
In the paper, a preliminary version of the methodology for determination of default dispersion measures for RC frames is developed. In order to reduce the computational work, a simplified two-step procedure for determination of dispersion measures is proposed. In the first step, statistical characteristics of global system parameters are determined by means of nonlinear static (pushover) analysis and consecutive idealization of pushover curves. In the second step, total dispersion of response is determined by Incremental Dynamic Analyses (IDA) for a group of equivalent SDOF systems and a group of ground motion records. The characteristics of SDOF structural models are generated based on the data from the first step. The effectiveness of the proposed approach and recently developed practice-oriented approach for probabilistic seismic assessment of building structures is tested on the selected result of a four storey building. Good agreement between the results obtained by both simplified approaches and results obtained by more "accurate" procedures is observed.

Keywords: Simplified nonlinear analysis, probabilistic assessment, dispersion measures, N2 method

1. INTRODUCTION

Earthquake engineering is characterized by large aleatory (random) and epistemic uncertainty (knowledge). As a result, the goals of performance-based earthquake engineering can be achieved only within probabilistic seismic assessment. A comprehensive framework for probabilistic performance assessment has been developed at the Pacific Earthquake Engineering Research (PEER) Center (Cornell, Krawinkler, 2000; Deierlein, 2004). One of the methods developed within the PEER probabilistic methodology is the SAC-FEMA method, which permits probabilistic assessment in close form (Cornell et al., 2002). Recently, a practice-oriented probabilistic approach for seismic performance assessment of building structures was proposed (Fajfar, Dolsek, 2012) which will be called in this paper the "Probabilistic N2" method. The approach combines the SAC-FEMA method, with the pushover-based N2 method (Fajfar, 2000). The most demanding part of the PEER probabilistic framework, that is the Incremental Dynamic Analysis (IDA) (Vamvatsikos, Cornell, 2002), is replaced by much simpler Incremental N2 method (IN2) (Dolsek, Fajfar, 2007), which requires considerably less input data and much less computational time, but which can, nevertheless, often provide acceptable estimates for the mean values of the structural response. The IN2 approach is not intended for the determination of dispersion, therefore default values for dispersion measures for aleatory and epistemic uncertainty have to be used for simplified risk assessment.

The aim of the study reported in this paper is development of default dispersion measures for typical reinforced concrete (RC) structural systems, which can be used in the Probabilistic N2 method or in any other method which is based on simple nonlinear methods. Both aleatory and epistemic uncertainties, i.e. information on β-values at the level of the structure, are being investigated using extensive numerical studies. Due to large extend of proposed numerical studies, a simplified two-step procedure for the determination of dispersion measures for RC frames was developed. The main idea of the proposed approach is to determine approximate IDA curves for equivalent SDOF systems, which allow a significant reduction of computational time. Similar approach was used by Dolsek (2011) for simplified risk assessment of RC frames with consideration of aleatory and epistemic uncertainty, where the author applied the previously developed methodology (Dolsek, 2009) to an equivalent SDOF structural model. Simplified nonlinear methods were used also by developers of IDA (Vamvatsikos, Cornell, 2002) for the development of the software tool SPO2IDA (Vamvatsikos, Cornell, 2006), which permits the estimation of approximate IDA curve based on nonlinear static (pushover) analysis results.

In the paper, a preliminary version of the methodology for determination of default dispersion measures for RC frames is developed and the effectiveness of the proposed approach is tested on selected result of a four storey building. The Probabilistic N2 method, in combination with determined values of dispersion, is used for approximate estimation of seismic risk.

2. METHODOLOGY

In this section, the Probabilistic N2 method for risk assessment of buildings proposed in (Fajfar, Dolsek, 2012) is briefly summarized and the proposed methodology for determination of the dispersion measures of structural response is explained.

2.1. Probabilistic N2 method for Risk Assessment of Buildings

The mean annual probability of exceeding a selected limit state can be approximately determined using Eqn. 2.1, which represents an extended form of the equation proposed in (Fajfar, Dolšek, 2012). In the extended form, “failure” is replaced by a selected limit state, and peak ground acceleration at “failure” PGA_C, is replaced by spectral acceleration at the fundamental period of the building, corresponding to a selected limit state, $S_{a,LS}$. (Note that “failure” in (Fajfar, Dolšek, 2012) corresponds to the near collapse (NC) limit state.)

$$P_{LS} = \exp\left[0.5\, k^2\, \beta^2_{TOT,LS}\right] H(S_{a,LS}) = \exp\left[0.5\, k^2\, \beta^2_{TOT,LS}\right] k_0\, S_{a,LS}^{-k} \tag{2.1}$$

The first factor in Eqn.2.1 represent the amplification factor of seismic risk, which is related to the total dispersion of response for selected limit state, $\beta_{TOT,LS}$. The second factor represents the mean value of the seismic hazard function at the spectral acceleration for a selected limit state $S_{a,LS}$, or, in other words, at the seismic intensity, expressed in terms of spectral acceleration, at which the selected limit state is attained. $H(S_{a,LS})$ can be approximated in a closed form as $k_0\, S_{a,LS}^{-k}$ .The parameters k_0 and k represent the intercept and the slope of the hazard function in log-log domain. In the Probabilistic N2 method, the value of the spectral acceleration for selected limit states, $S_{a,LS}$, is determined by means of the N2 method.

2.2. Simplified Approach for Determination of Dispersion Measures for RC Building

For practical application of Eqn. 2.1 predetermined values of dispersion for typical structural systems should be available. In order to reduce the computational work to a reasonable level, a simplified two-step approach has been developed for the determination of $\beta_{TOT,LS}$ for different structural systems.

Following the main idea and limitations of the basic N2 method, nonlinear static (pushover) analyses and transformations from MDOF to SDOF systems are performed in the first step of the proposed approach (Fig. 2.1). Epistemic uncertainty is introduced through the analysis of a group of structural models, which are generated by Latin Hypercube Sampling (LHS) method (Vorechovsky, Novak, 2003). Most important local sources of dispersion of a MDOF system, i.e. dispersion of parameters at the element or storey level, are taken into account, such as storey masses m_i, element concrete strength f_{cm}, element steel yield stress f_{sy}, effective slab widths b_{eff}, initial stiffness Θ_y, and ultimate rotation Θ_{nc} of elements (Fig. 2.1). The main idea of the proposed approach is to reduce the number of uncertain structural parameters by linking the local sources of dispersion with the dispersion of the parameters of the global system, such as system stiffness, strength and ductility. The global system properties should be expressed with parameters that allow comparison between different structural systems. Following this idea, the following global system parameters are chosen in the preliminary version of the procedure: the system strength is expressed in terms of acceleration capacity S_{ay}, the system stiffness in terms of elastic period of the system T, and the system ductility in terms of the ductility at the beginning of degradation μ_m and ductility at collapse μ_u. Additional global system parameters such as mass of equivalent SDOF system m^* and transformation factor Γ are also employed in the procedure. The relation between the dispersion at the local and the global level is obtained by pushover analyses and idealization of pushover curves. Reduction of structural parameters is a result of the pushover analysis, through which the structural behaviour of a complex MDOF system is summarized with a global force-displacement relationship, which is, in the proposed procedure, idealized by a simple tri-linear backbone curve (Fig. 2.1). The idealization procedure provides, for main force-displacement parameters of the system, i.e. system yield strength F_y, yield displacement u_y, displacement at the beginning of degradation u_m, and displacement at collapse u_u, a sample of random values, which are then used to determine the most appropriate probability distribution for global system parameters. The relations between the two sets of parameters are defined as follows:

$$S_{ay} = \frac{F_y}{\Gamma\, m^*} \qquad T = 2\pi\sqrt{\frac{m^*\, d_y}{F_y}} \qquad \mu_m = \frac{u_m}{u_y} \qquad \mu_u = \frac{u_u}{u_y} \tag{2.2}$$

Kolmogorov-Smirnov distribution test is used to assess the appropriateness of assumed probability distributions. Correlation between global system parameters is also determined, since it is needed in the second step of the procedure.

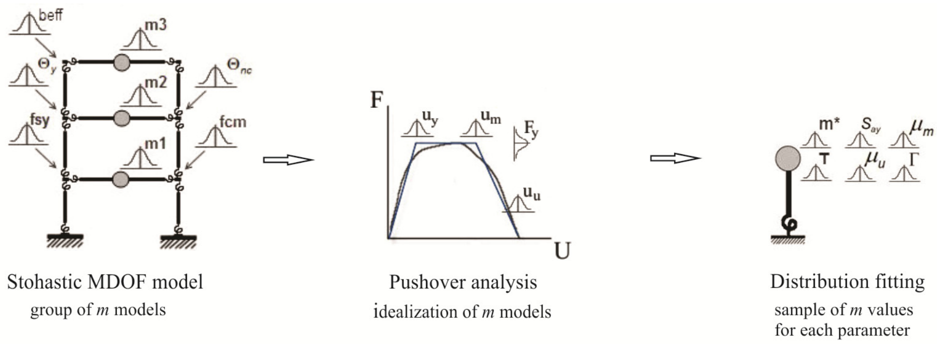

Figure 2.1. Schematic representation of the first step of the proposed methodology

In the second step, Incremental Dynamic Analyses (IDA) of equivalent SDOF systems (SDOF-IDA) are performed, and the value of total dispersion, related to both aleatory and epistemic uncertainty, is determined by the analysis of a group of models and a group of ground-motion records (see Fig. 2.2). The characteristics of SDOF structural models are generated based on the probability distributions and correlations of global system parameters, which were determined in the first step of the procedure. The use of SDOF systems instead of MDOF systems allows a significant reduction of computational time.

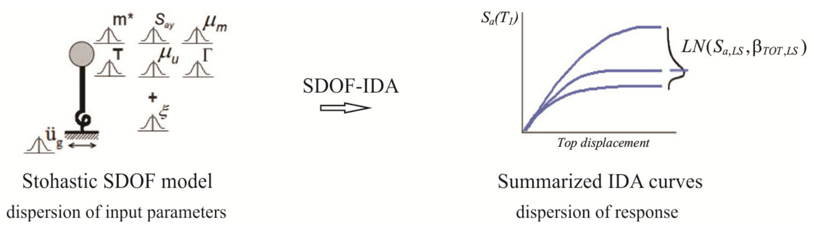

Figure 2.2. Schematic representation of the second step of proposed methodology

LHS method is employed for the generation of a group of structural models. Since this step of the procedure employs nonlinear dynamic analysis, additional uncertainties related to the hysteretic behaviour of structure and damping need to be introduced. In the preliminary stage of the proposed methodology, only uncertainty related to the damping of the system was taken into account. The hysteretic rules were assumed to be fixed.

In SDOF-IDA the definition of limit states for each system is usually based on results obtained in the pushover analysis of the corresponding MDOF system. In the proposed approach, such a definition of limit states is not appropriate, since the pushover results from the first stage cannot be directly linked to the generated SDOF systems. In the preliminary version of the proposed methodology, this problem was resolved by introducing the parameter r_{LS} which determines the relative position of limit states between the ductility of 1 and the ductility at collapse μ_c. The parameter r_{LS} has a value of zero if a limit state occurs at ductility of 1, and value of 1 if a limit state coincides with the ductility at failure μ_c. For an individual model, the parameter $r_{LS,i}$ is calculated based on results obtained in pushover analysis, as the ratio of ($\mu_{LS,i} - 1$) and ($\mu_{c,i} - 1$), where $\mu_{LS,i}$ and $\mu_{c,i}$ represents the ductility at selected limit state and the ductility at collapse, respectively. For the definition of limit state of generated SDOF systems, a common value of the parameter r_{LS} is used for all models and is determined as the average value of ratios $r_{LS,i}$ for individual models.

The proposed procedure will be used in an extensive study of typical structural systems aimed at determining predetermined values for dispersion due to aleatory and epistemic uncertainty. In this paper, only one example is shown in the next chapter. Of course, the proposed procedure is, like any simplified approach, subjected to several limitations. It should be noted that the limitations of the proposed procedure are consistent with the limitations of the basic N2 method (Fajfar, 2000). Both procedures are, in principle, inaccurate for structures with significant higher mode effects.

3. CASE STUDY: A FOUR-STOREY RC FRAME BUILDING

3.1. Description of the Four-Storey RC Frame Building and Mathematical Modelling

The presented methodology is applied for a case study of a four-storey planar RC frame, which was pseudo-dynamically tested in full scale at ELSA laboratory. The elevation, plan and reinforcement in columns of the structure are presented in Fig. 3.1. The structure had been designed to reproduce the design practice in European and Mediterranean countries about half century ago. Additional information about the structure and experimental results can be found in (Carvalho, Coelho, 2001).

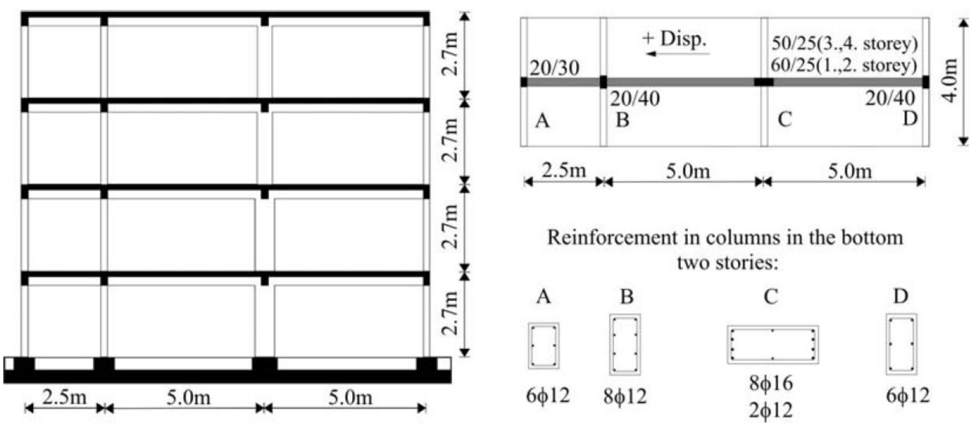

Figure 3.1. The elevation, plan view, and typical reinforcement in columns of four-storey RC frame

The nonlinear analyses of the structure were performed with OpenSees (McKenna et al., 2004), using the performance-based engineering (PBEE) toolbox (Dolsek, 2010), which represents a simple yet effective tool for seismic performance assessment of RC frames. The beam and column behaviour is modelled by one-component lumped plasticity elements, composed of an elastic beam and two inelastic rotational hinges at both ends. The moment-rotation relationship is modelled in accordance to previous studies (Fajfar et al., 2006). OpenSees hysteretic material is used for the definition of the hysteretic behaviour of both MDOF and SDOF systems. The ultimate rotation in the columns at near collapse (NC) limit state, which corresponds to 20% reduction in maximum moment, is estimated by means of the conditional average estimator (CAE) method (Perus et al., 2006). For beams, the EC8-3 (CEN, 2005) formula is used and the parameter γ_{el} is assumed to be equal to 1.0. Due to the absence of seismic detailing the ultimate rotations are multiplied by factor of 0.825 (CEN, 2005). The reader is referred to (Dolsek, 2010) for additional information about modelling assumptions.

The limit state terminology applied in the modelling of the structure is used according to Eurocode 8 provisions (CEN, 2005). Three limit states are analysed: damage limitation (DL), significant damage (SD) and near collapse limit state (NC). The following definitions of limit states are employed in the nonlinear analyses of the MDOF structure. The limit state DL is assumed to occur when the maximal story drift exceeds 0.5 %. The limit states SD and NC are assumed to take place when the first column riches maximum moment, and when the maximum moment of the first column reduces for 20 %, respectively. The definition of limit states for generated SDOF systems are determined in accordance with the methodology presented in section 2.2.

3.2. Ground Motion Selection

A group of 30 ground-motion records was selected from the PEER Next Generation Attenuation (NGA) strong-motion database (Chiou et al., 2008). The magnitude of earthquakes ranges from 5.5 to 7.5 and the distance to source from 5 to 50 km. A ground motion selection algorithm, developed within the Baker research group at Stanford University, is employed (Jayaram et al., 2011). The procedure allows computationally efficient and theoretically consistent selection of site and structure specific ground motions that match the target response spectrum mean and variance. The Eurocode 8 (CEN, 2004) target spectrum corresponding to peak ground acceleration of 0.2 *g* and soil type B is used. Based on the study of the attenuation relationship developed by Campbell and Bozorgnia (2008) a constant value of dispersion 0.6 is assumed for the target spectrum. The acceleration spectra for selected ground-motion records, scaled to spectral acceleration at the fundamental period of the structure T=0.92 s, along with their fractile values and target spectra, are presented in Fig. 3.2.

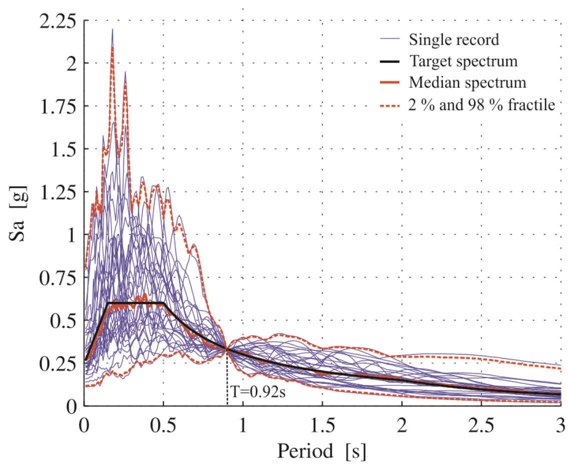

Figure 3.2. Acceleration spectra for selected ground motions, scaled to spectral acceleration at the fundamental period of the test structure, target spectra and fractiles

3.3. Input Random Variables, Statistical Correlation and Sampling

A group of 30 models, generated by Latin Hypercube Sampling (LHS) method, is used to simulate the statistical properties of structural parameters. In the case study, most important sources of the uncertainty of MDOF systems parameters are taken into account (Table 3.1). Spatial variability of uncertainty is neglected. Most of the input variables are modelled as uncorrelated. Exception is the correlation between the yield and near collapse rotations and the correlation between the effective slab widths of shorter and longer beams, which are taken as 0.8 and 0.5, respectively. The statistical characteristics of the input random variables presented in Table 3.1 are taken from (Dolsek, 2009), with the exception of the mean value of damping, which amounts to 5 % instead of 2 %.

Table 3.1. Statistical characteristics of the input random variables

Name		Mean or Median	COV	Distribution
mass - top, other stories	m_t, m_i	40 t, 46 t	0.10	normal
concrete strength	f_{cm}	16 MPa	0.20	normal
steel yield stress	f_{sy}	343.4 MPa	0.05	lognormal
effective slab width	b_{eff}	75 or 125 cm	0.20	normal
initial stiffness of the columns	$\Theta_{y,c}$	1·computed	0.36	lognormal
initial stiffness of the beams	$\Theta_{y,b}$	1·computed	0.36	lognormal
ultimate rotation of the columns	$\Theta_{u,c}$	1·computed	0.40	lognormal
ultimate rotation of beams	$\Theta_{u,b}$	1·computed	0.60	lognormal
system damping	ξ	5 %	0.40	normal

3.4. Determination of Dispersion Measures for Selected Limit States

The dispersion measures for selected limit states are determined using the methodology described in section 2. In the first step, nonlinear static (pushover) analyses of a group of 30 models are performed. In the pseudo-dynamic test in Ispra, a storey mechanism in the third storey was observed. The same plastic mechanism is predicted by the pushover analysis of the deterministic model. The pushover results, presented in Fig. 3.3, reveal a large dispersion of response of the test structure. A detailed analysis of plastic mechanisms suggests that the large dispersion of response is result of a change in failure modes of the test structure due to model uncertainty. To facilitate the distinction between different plastic mechanisms, different colours are used in Fig. 3.3.

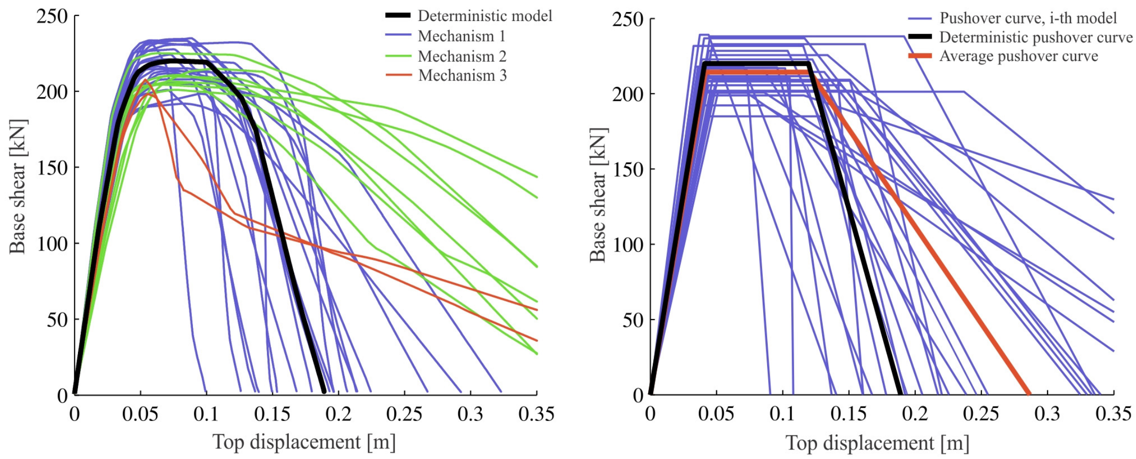

Figure 3.3. Pushover curves and their idealizations for 30 realizations of the stochastic model

Three distinct plastic mechanisms are observed. In most cases, i.e. in 20 cases out of 30, the same failure mechanism as in the deterministic model is observed (mechanism 1). In 8 cases, a more favourable mechanism along the first three storeys is predicted (mechanism 2). In two remaining cases, a more pronounced drop of capacity, due to small rotation capacities of the beams in comparison with columns, preceded the final plastic mechanism over three storey (mechanism 3). A high sensitivity of the test structure to modelling assumptions was noticed also in the case of the deterministic model. If the EC8-3 (CEN, 2005) provisions were used for the determination of the rotation capacity of the columns, a different plastic mechanism than that observed in the pseudo-dynamic test occurred.

Equal energy (area) concept is employed for the idealization of pushover curves with a tri-linear backbone relationship (Fig. 3.3). The statistical characteristics of global system parameters, i.e. the system acceleration capacity S_{ay}, the elastic period of the system T, the ductility at the beginning of degradation μ_m, the ductility at collapse μ_u, the mass of the equivalent SDOF system m^*, and the transformation factor from the MDOF to the SDOF system Γ, are determined in accordance with section 2.2 and are presented in Table 3.2. The results presented in Fig. 3.3 (right) and Table 3.2 suggest that there is a difference between the deterministic and the average pushover curve for all models. The major difference is observed in the case of the post-capping branch of the idealized pushover curve, which is most influenced by the formation of a different plastic mechanism. Smaller differences are observed in case of the system stiffness and capacity, which result in a change of the fundamental period from 0.92 to 0.94 s and system capacity from 0.151 to 0.147 g, for the deterministic and the average system model, respectively.

Table 3.2. Statistical characteristics (left) and correlation matrix (right) for the global system parameters

Name	Mean	COV	Distribution
S_{ay}	0.147 g	0.08	normal
T	0.94 s	0.08	normal
μ_m	2.90	0.28	normal
μ_u	6.16	0.51	lognormal
m^*	115.8 t	0.05	normal
Γ	1.29	0.01	normal

	S_{ay}	T	μ_m	μ_u	m^*	Γ
S_{ay}	1	-0.67	0	-0.29	-0.64	0.34
T	-0.67	1	0.04	0.27	0.40	-0.20
μ_m	0	0.04	1	0.20	0	-0.14
μ_u	-0.29	0.27	0.20	1	0.06	0.31
m^*	-0.64	0.40	0	0.06	1	-0.25
Γ	0.34	-0.20	-0.14	0.31	-0.25	1

Kolmogorov-Smirnov distribution test “confirmed”, that normal distribution is appropriate for most parameters, expect for the collapse ductility μ_u, which is fitted with a lognormal distribution. Graphical representation of the global system parameter, along with the idealization rule, is presented in the case of the pushover result for the deterministic model (Fig. 3.4). The limit states of the deterministic model are determined in accordance with section 3.1.

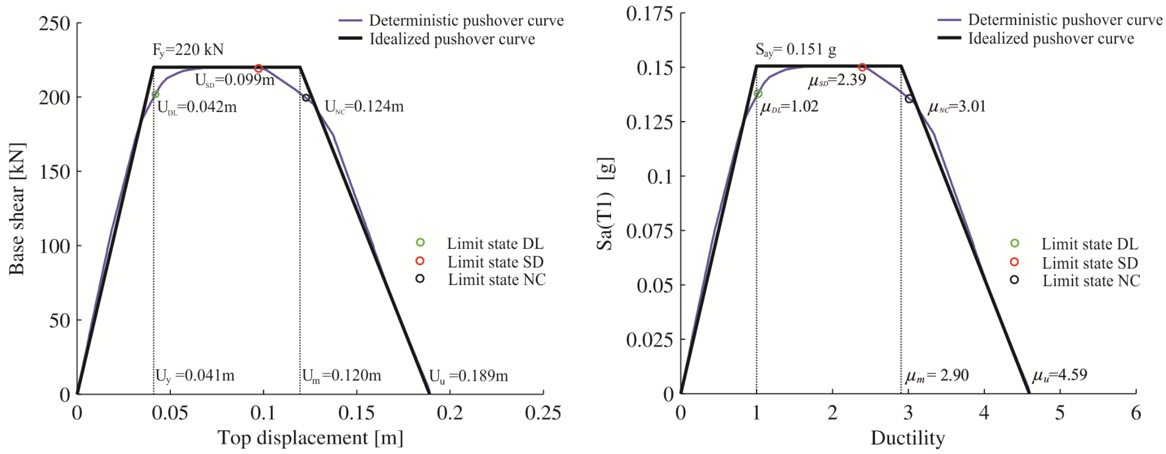

Figure 3.4. Idealization of the deterministic pushover curve, dimensional (left) and dimensionless (right) parameters

Statistical characteristics determined in the first step of the procedure (Table 3.2) are used in the second step to assemble a stochastic SDOF model for the structure. To simulate the statistical properties of the stochastic model, a group of 30 SDOF systems is generated by means of the LHS method. IDA is performed for the group of 30 ground-motion records (section 3.2) and for the generated SDOF models to incorporate both sources of uncertainty. The comparison of approximate SDOF-IDA and “accurate” IDA curves, determined by IDA of the MDOF systems (MDOF-IDA), is presented in the left part of Fig. 3.5. In general, a good agreement between the results obtained by both approaches is observed. The effectiveness of the proposed approach decreases in the vicinity of the global dynamic instability.

Based on the SDOF-IDA results (left part in Fig. 3.5), total dispersion of seismic response for different limit states of the test structure is determined. As explained in section 2.2, common values of parameters r_{DL}=0.01, r_{SD}=0.25 and r_{NC}=0.43, which represents the relative position of limit states between the ductility of 1 and the ductility at collapse μ_c , are used in the proposed approach for the definition of the limit states. Median and standard deviation of natural logarithms of structural response, in terms of the first mode spectral acceleration at the selected limit state, $S_{a,LS}$ and $\beta_{TOT,LS}$ respectively, are presented in Table 3.3. The results of the proposed approach are compared with the MDOF-IDA results.

Table 3.3. Comparison of the structural response determined by the proposed approach and MDOF-IDA

	Proposed approach		MDOF- IDA	
Limit state	$S_{a,LS}$ (g)	$\beta_{TOT,LS}$	$S_{a,LS}$ (g)	$\beta_{TOT,LS}$
DL	0.15	0.150	0.14	0.202
SD	0.32	0.344	0.31	0.300
NC	0.41	0.400	0.42	0.340
C	0.57	0.464	0.57	0.397

The results presented in Table 3.3, show good agreement of both approaches for all calculated limit states. Slightly conservative estimates for the dispersion of response are determined with the proposed approach, whereas the estimates of the median values are very accurate. The difference between dispersion measures estimated by the proposed approach and "accurate" values are in the range of +15 %, except for the limit state DL, where a larger difference is observed (−25 %). A smaller dispersion for the limit state DL, determined by the proposed procedure, is related to the fact that the SDOF system, in contrast to the actual MDOF system, cannot detect the elastic dispersion of response related to higher mode effects. Dispersion measures, determined by the proposed approach, cannot be treated as default values, since they are determined for the presented example only. However, employing the simplified methodology for a group of benchmark frame structures default values for dispersion measures can be determined. In comparison to the dispersion measure obtained from the data in the literature (β_{TOT}=0.6) employed in (Fajfar, Dolsek, 2012) for the estimation of "failure" probability, i.e. the probability of exceeding near collapse (NC) limit state according to the terminology used in this paper, a smaller value for the total dispersion of response at the equivalent limit state level is determined in the paper.

3.5. Risk Assessment of the Test Building by the Probabilistic N2 method

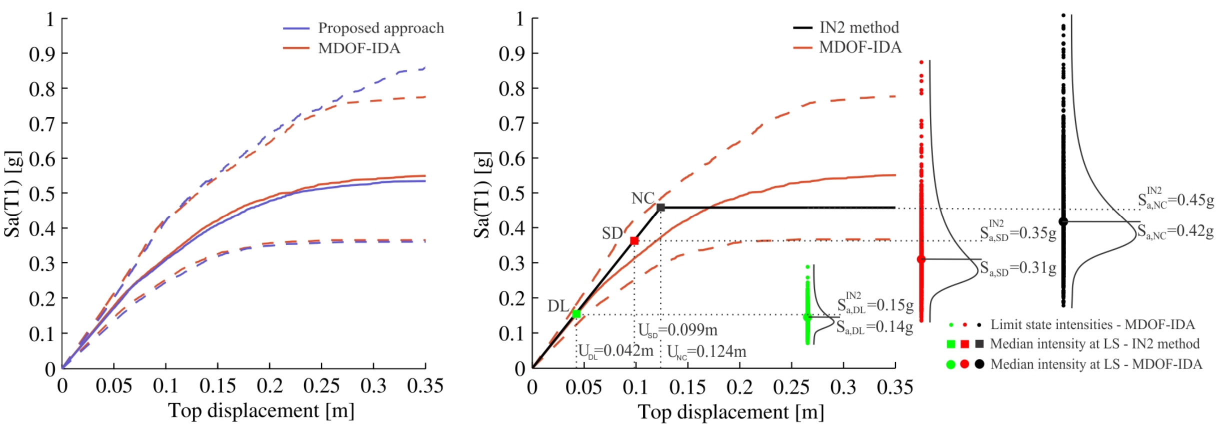

Figure 3.5. Comparison of fractile curves (16,50,84) between the proposed approach (SDOF-IDA) and MDOF-IDA (left), and median intensities at three limit states between IN2 method and MDOF-IDA (right)

Dispersion measures determined in the section 3.4 are used in Eqn. 2.1 for the risk assessment of the test structure. It is assumed that the structure is located in a moderate seismic region on soil type B, with the peak ground acceleration for a 475-years event of 0.25 *g*. The parameters of the seismic hazard function amount to k=3 and k_0=1.45·10^{-4}. In the first step, the spectral accelerations $S_{a,LS}$ "corresponding" to the displacement capacities U_{LS} at three limit states, which were previously determined by the pushover analysis of the deterministic model, are determined by means of the N2 method. Since the "equal displacement rule" is assumed, the IN2 curve is completely defined by a single point corresponding to the NC limit state. Note that in the Probabilistic N2 method, the NC limit state is conservatively assumed as the "failure" of the building. The IN2 curve, along with the median values of spectral accelerations at selected limit states, $S_{a,LS}^{IN2}$ and $S_{a,LS}$, determined by the N2 method, and MDOF-IDA results are presented in the right part of Fig. 3.5. Mean annual probability of exceeding the selected limit states P_{LS} are presented in Table 3.4, together with the probabilities of exceeding limit states in 50 years P_{LS}^{50}, which are determined as $1-(1-P_{LS})^{50}$.

Table 3.4. Mean annual probabilities and probabilities of exceeding selected limit states in 50 years

Probabilistic N2 approach					
Limit state	$S_{a,LS}$ (g)	$\beta_{TOT,LS}$	$H(S_{a,LS})$	P_{LS}	P_{LS}^{50} [%]
DL	0.15	0.150	$4.77\cdot10^{-2}$	$5.28\cdot10^{-2}$	93
SD	0.35	0.344	$3.76\cdot10^{-3}$	$6.41\cdot10^{-3}$	27
NC	0.45	0.400	$1.75\cdot10^{-3}$	$3.59\cdot10^{-3}$	16

"Accurate" approach based on MDOF-IDA					
Limit state	$S_{a,LS}$ (g)	$\beta_{TOT,LS}$	$H(S_{a,LS})$	P_{LS}	P_{LS}^{50} [%]
DL	0.14	0.202	$5.30\cdot10^{-2}$	$6.37\cdot10^{-2}$	96
SD	0.31	0.300	$5.36\cdot10^{-3}$	$8.04\cdot10^{-3}$	33
NC	0.42	0.340	$2.22\cdot10^{-3}$	$3.75\cdot10^{-3}$	17

A good agreement of probabilities of exceeding three limit states is observed between simplified and more accurate analysis procedures. Approximate results are slightly unconservative for all limit states. One of the reasons may be the treatment of epistemic uncertainty which, according to Dolsek (2011), does not only increase the dispersion of response, but also decreases the limit state intensities. Applying the terminology used in (Fajfar, Dolsek, 2012), a 16 % probability of "failure" (i.e. the NC limit state) in the lifetime of the structure is determined by the Probabilistic N2 method. Larger probability of "failure", i.e. 38 % in lifetime of the structure, was determined in (Fajfar, Dolsek, 2012), since the assumed total dispersion of response was larger than the dispersion of response determined in this paper.

4. CONCLUSIONS

In the paper, a preliminary version of a two-step procedure for the determination of dispersion measures for simplified probabilistic assessment of reinforced concrete (RC) frames is proposed. It is intended to be used for the determination of the default dispersion measures for typical structural systems. The approach allows a significant reduction of computational time and of the number of uncertain structural parameters. Assuming that the dispersion due to aleatory uncertainty can be properly assessed by a representative group of ground-motion records, the total dispersion of structural response is related only to the dispersion of global system parameters. Thus, the total dispersion of response can be expressed as a function of statistical properties of global systems parameters, such as system capacity, period and ductility.

As a test example, the proposed approach was applied to a four-storey planar RC frame. For the test example, good agreement of dispersion measures determined with the proposed approach and a more accurate MDOF-IDA approach was observed. Moreover, an assessment of the test building by means of the recently proposed Probabilistic N2 method was made. Comparison of seismic risks for three limit states reveals quite a good agreement between the results obtained by the simplified approach and those obtained by more accurate analysis using IDA of MDOF systems.

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