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OPTIMAL PERFORMANCE-BASED SEISMIC DESIGN OF STRUCTURES USING APPROXIMATE PERFORMANCE ESTIMATION METHODS

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Abstract. *We propose a new approach for the performance-based seismic design of buildings using a deterministic and a reliability-based structural optimization framework. To overcome the increased computing cost of Incremental Dynamic Analysis (IDA) we adopt an approximate seismic performance estimation tool, known as Static Pushover to IDA (SPO2IDA). The SPO2IDA tool is nested within the framework of a Genetic Algorithm resulting to an efficient seismic design procedure able to consider uncertainty. The Genetic Algorithm steps towards designs of improved performance, locating the most efficient design in terms of the minimum weight of the structure. Reliability-based constraints are considered in terms of the mean annual frequency of preset limit-states not being exceeded. A three-storey steel moment resisting frame is used to demonstrate the design algorithm proposed. The methodology presented leads to efficient real-world building designs within reasonable computing time, directly considering the seismic risk.*

1 INTRODUCTION

The design of a building structure is valued upon the extent to which regional design codes or guidelines are satisfied. This effort comes in hand with the designer's need for reducing the cost in order to obtain a more efficient design solution. Nonlinear, static or dynamic, methods of analysis are expected to lead to less costly designs using high-level performance criteria, since the engineer is allowed to have a better insight on the system's demand and capacity. Among the nonlinear performance estimation methods today available Incremental Dynamic Analysis (IDA) [1] is probably the most powerful and thorough approach.

IDA as well as other nonlinear methods of analysis requires increased computing resources. For practical applications and for designing new structures, approximating methods based on IDA have recently appeared intending to provide a fast alternative to the original method. More specifically, Dolsek & Fajfar [2] proposed the IN2 method, which is a simplified procedure that combines nonlinear static analysis with a design response spectrum. Vamvatsikos & Cornell [3,4] developed the Static Pushover to Incremental Dynamic Analysis (SPO2IDA) tool in an effort to approximate the IDA curve taking advantage information extracted from the static pushover backbone. Han & Chopra [5] proposed the MPA-based IDA method which in essence is a variation of the Modal Pushover Analysis (MPA) method. Azarbakht & Dolsek [6] proposed a method that uses a limited number of ground motions that have been appropriately selected to obtain the mean and the fractiles of the response. All the above procedures are approximate, but their results compare sufficiently to those of IDA while their cost and efficiency varies.

Evolutionary-based optimizers can handle complicated structural problems at the expense of more optimization cycles. Their rapid development made possible the solution of complex and realistic nonlinear structural optimization problems. Evolutionary Algorithms (EA) do not require the calculation of gradients of the constraints, as opposed to mathematical programming algorithms, and thus structural design code checks can be implemented as constraints of the optimization problem in a straightforward manner. Within this concept, different optimization-based seismic design procedures can be developed (Fragiadakis & Lagaros [7]).

This study discusses the use of approximating performance-estimation methods within the framework of an optimization algorithm. The resource-demanding IDA method is replaced by the Static Pushover to Incremental Dynamic Analysis (SPO2IDA) approach. SPO2IDA is employed to provide fast estimates of the mean and the dispersion of the demand at various performance levels. The constraints of the optimization problem are introduced as the exceedance of every performance level calculated either using deterministic or probabilistic design criteria. Deterministic design criteria refer to the exceedance of preset drift values that are set by the codes. Probabilistic criteria refer to the calculation of the mean annual frequency of exceedance of every limit state considering in this way the uncertainties. A Genetic Algorithm is used to handle the resulting optimum design problem. A three-storey, steel moment-resisting frame (SMRF) is used to demonstrate the proposed methodology.

2 STRUCTURAL PERFORMANCE ESTIMATION

2.1 Incremental Dynamic analysis

According to the Incremental Dynamic Analysis (IDA) method [1] the mathematical model of the structure is subjected to a suite of ground motion records incrementally scaled to different levels of seismic intensity. Recent researches show that the scaling practice is legitimate and introduces slight bias on the prediction of the structural response [10]. The building's capacity is visualised with a curve of an Engineering Demand Parameter (EDP), e.g. maximum interstorey drift ratio, versus an Intensity Measure (IM), e.g. the 5%-damped, first-mode spectral acceleration $S_a(T_1,5\%)$, representing the seismic intensity. IDA allows calculating the median (50% fractile) and also the dispersion (16%, 84% fractiles) of the building's capacity. Performance limit-states are defined on these curves by appropriate limits which are set, preferably, on the EDPs. The results of IDA can be easily combined with probabilistic seismic hazard analysis in order to estimate the mean annual frequency (MAF) of a limit-state being exceeded.

2.2 Static Pushover To Incremental Dynamic Analysis (SPO2IDA)

The Static Pushover to IDA (SPO2IDA) tool [3, 4] provides an approximate estimation of the IDA curve using the backbone of the static pushover (SPO). SPO2IDA has been verified for numerous SDOF systems and first-mode dominated structures and can be seen as a more elaborate R-μ-T relationship. More specifically, the static pushover is approximated with a trilinear or a quadrilinear envelope in order to extract the parameters that describe the SPO curve (Fig. 1). The extracted parameters are then given as input to the SPO2IDA tool to provide the fractile IDAs in normalized coordinates of the strength reduction factor R versus the ductility μ. The final approximate IDAs are obtained after a series of calculations on the available R-μ data [11].

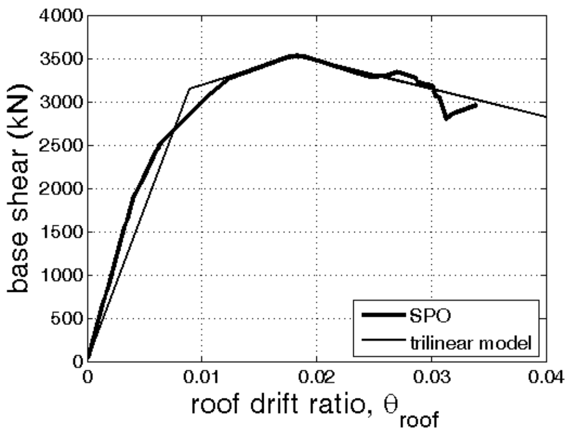


Figure 1. The SPO curve for the three-storey steel moment resisting frame and its approximation with a trilinear model.

The necessary steps to obtain the approximate IDAs are briefly summarised as follows. The process begins with approximating the static pushover curve with a multilinear envelope. Having approximated the SPO capacity curve with a trilinear model, as can be seen in Figure 1, the parameters that describe the backbone can be easily extracted. These parameters refer to the properties of the backbone curve, which initially allows for elastic behavior up to F_y, then hardens at a non-negative normalized slope of a_h while beyond this point, a negative stiffness segment starts having a normalized slope a_c [11]. These parameters are given as input to SPO2IDA to produce the median capacities. Since the capacities of SPO2IDA are in dimensionless R-μ coordinates, they have to be scaled to another pair of IM-EDP coordinates, such as the 5%-damped, first mode spectral acceleration, $S_a(T_1,5\%)$, and the maximum interstorey drift ratio, θ_{max}.

The scaling from R-μ to $S_a(T_1,5\%)$-θ_{max} is performed with simple algebraic calculations:

$$\mathbf{S_a}(T_1,5\%)=\mathbf{R}\,S_a^{yield}(T_1,5\%)$$
$$\boldsymbol{\theta}_{roof}=\boldsymbol{\mu}\,\theta_{roof}^{yield} \tag{1}$$

where $\boldsymbol{\theta}_{\mathrm{roof}}$ is the roof drift (displacement of the roof divided by the building height) and θ_{roof}^{yield} is the roof drift at yield, respectively. Once θ_{roof}^{yield} is known, $\boldsymbol{\theta}_{\max}$ can be extracted from the results of the SPO, since for every load increment the correspondence between the two EDPs is always available. To determine $S_a^{yield}(T_1,5\%)$ and θ_{roof}^{yield} we assume that the yield roof drift is that of the trilinear approximation, while the $S_a^{yield}(T_1,5\%)$ is related to the approximation of the elastic "stiffnesses" (or slopes) of the median IDA curves plotted with θ_{roof} as the EDP. The stiffness, denoted as k_{roof}, is the median value obtained using elastic response history analysis with a few ground motion records, or alternatively by using standard response spectrum analysis. An approximate relationship for k_{roof} can be found in [11]. Finally, $S_a^{yield}(T_1,5\%)$ will be:

$$S_a^{yield}(T_1,5\%)=k_{roof}\,\theta_{roof}^{yield} \tag{2}$$

In summary, the process of producing an approximate IDA curve from a single static pushover run involves the following steps. Initially perform a static pushover analysis with a first-mode lateral load pattern and then approximate it with a trilinear model. Next SPO2IDA will provide the IDA curves in normalized R-μ coordinates which have to be transformed in terms of $S_a(T_1,5\%)$ versus $\theta_{\max}$. This requires the elastic slope of the actual IDA, k_{roof} when θ_{roof} is the EDP. With the aid of Equations (1) and (2) we obtain the IDAs in $S_a(T_1,5\%)$-θ_{roof} coordinates. The final IDAs are obtained using the mapping between θ_{roof} and $\theta_{\max}$, available from the results of the static pushover. For a three-storey SMRF building the computing time comes down from 1.5÷2 hours required for a single IDA, to a couple of minutes, approximately two orders of magnitude less.

3 METHODOLOGY-ALGORITHM

The aim of sizing optimization problems is to minimize an objective function which, usually, is proportional to the cost of the structure. The most common objective function for steel structures is the total weight, which is considered to be directly related to the cost. The design variables are chosen to be the cross sections of the members of the structure such that the objective function can be expressed as their linear, or nonlinear, combination. Due to engineering practice demands, the members are divided into groups of design variables, thus providing a trade-off between the use of more material and the need for symmetry and uniformity due to practical considerations. Moreover, due to fabrication limitations, the design variables are not continuous but discrete. A discrete deterministic-based structural optimization (DBO) problem is formulated as follows:

$$\begin{aligned}&\min\ F(\boldsymbol{s})\\&\text{s.t.}\begin{cases}g_i(\mathbf{s})\ge 0,\ i=1,\dots,l\\ s_j\in R^d,\ j=1,\dots,m\end{cases}\end{aligned} \tag{3}$$

where $F(\boldsymbol{s})$ is the objective function to be minimized and g_{i} are the l deterministic constraints. R^d is a given set of discrete values, and s_j is the design variables that can take values from this set. In a similar way, a discrete reliability-based (RBO) structural optimization problem is formulated as:

$$\min\ F(\boldsymbol{s})$$
$$\text{s.t.}\begin{cases} g_i(\boldsymbol{s}) \geq 0,\ i = 1,...,l \\ s_j \in R^d,\ j = 1,...,m \\ h_k(v_{\mathrm{EDP}}(\boldsymbol{s}) \leq v_{\mathrm{EDP}}^{\mathrm{lim}}(\boldsymbol{s})),\ k = 1,...,n \end{cases} \tag{4}$$

where $F(\boldsymbol{s})$ is the objective function to be minimized, R^d is a given set of discrete values, s_j represents the design variables that can take values from this set, h_k are the n probabilistic constraints, v represents the MAF of exceedance of the k_{th} performance level and finally EDP denotes a chosen engineering demand parameter (EDP) (here is the maximum interstorey drift $\theta_{\max}$).

3.1 Solving the optimization problem using Genetic Algorithms

Genetic Algorithm (GA) [12] is the most widely used Evolutionary Algorithm. GA is a machine-learning algorithm that uses a genetic metaphor and imitates the evolution of a population. The resulting numerical tool can be used for general purposes and does not need the calculation of gradients as traditional mathematical optimizers do. Implementations of GA, typically use fixed-length character strings (binary or real-valued) to represent their genetic information. Together with a population of individuals, which undergo mutation and crossover, the GA guides the search process towards the optimum combination of the design variables. The steps of the adopted GA-based algorithm are:

1. *Initialization step:* Random generation of an initial population of the vectors of design variables s^j (j=1,…, n_{pop}). The variables are encoded as binary strings.
2. *Analysis step (Fitness evaluation):* Firstly, perform checks that do not require analysis to ensure that the design complies with the "strong-column-weak-beam" philosophy and that other detailing requirements are met. Subsequently, perform linear elastic analysis to obtain the demand for the non-seismic load combinations and then perform Static Pushover for the seismic actions. Use the SPO2IDA tool to obtain the EDP or its mean annual frequency of exceedance for every limit-state considered. For every constraint that is violated, calculate the penalties and modify the objective function accordingly (section 3.2).
3. *Selection, Generation and Mutation step:* Apply the operators of GA to create the members of the next population t^j (j=1,…, n_{pop}).
4. *Final check:* If a prespecified number of generations has been reached stop, otherwise go back to step 2.

3.2 Performance-based earthquake engineering constraints

3.2.1 Design using deterministic criteria

For structural optimization problems under earthquake loading, the constraints adopted follow the performance-based design concept where the performance of the structure is evaluated at distinct levels of seismic intensity. Three performance levels are here considered: Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). Preliminary checks are performed on every candidate design. These checks include examining whether a soft storey mechanism produced by the hinges formed in the columns rather than the beams. Also, a check whether the sections chosen are of class 1, as EC3 suggests, is carried out. This check is important in order to ensure that the members are able to develop their full plastic moment and rotational ductility. Moreover, geometrical restrictions that ensure the correct connection of the beams to the columns are performed. Another check ensuring that the bending capacity of the beams is adequate against gravity loads is also carried out. If the checks are not satisfied the design is slightly modified in order to meet the above restrictions. The capacity of the structure against seismic loads is subsequently assessed. For the three limit-states considered the first-mode spectral acceleration is computed with the aid of the EC8 elastic response spectrum. Afterwards, we determine using SPO2IDA the maximum interstorey drift demand. The latter is compared to the drift threshold values of the corresponding limit-state.

When a performance criterion is violated, a penalty, *p*, is calculated which gives a measure of the deviation of the value obtained with analysis from the acceptable threshold value. In this work the objective function is penalised as follows:

$$\overline{\mathrm{F}}(\mathbf{s}) = \begin{cases} \mathrm{F}(\mathbf{s}), & \text{if } \mathrm{s} \in \mathrm{R}^{\mathrm{d}} \\ \mathrm{F}(\mathbf{s}) \cdot \max(p), & \text{otherwise} \end{cases} \tag{5}$$

where max(p) is the maximum value of the violated constraint's penalty parameter and $\overline{\mathrm{F}}(\mathrm{s})$ is the value of the penalized objective function. The selection of the penalty parameter is significant, since a large penalty will force the design procedure to work away from the region where the global optimum is located while a small penalty will make the algorithm converge to an infeasible solution. Moreover, the penalty parameter adjusts the weight of the penalty imposed on the objective function during the optimization process.

The penalty adopted for the i-th limit state of the deterministic-based formulation of the optimization algorithm takes the form:

$$p_{\theta\max} = \frac{\theta^{\mathrm{i}}_{\max} - \theta^{\mathrm{i}}_{\lim}}{\theta^{\mathrm{i}}_{\lim}} \tag{6}$$

where $\theta^{i}_{\max}$ and $\theta^{i}_{\lim}$ is the maximum interstorey drift demand and its threshold, respectively.

3.2.2 Design using probabilistic criteria

From another point of view, the use of probabilistic criteria posess the advantage of considering uncertainties through the use of probabilities. In this way, the seismic design checks are applied on the mean annual frequency of every limit-state instead of being applied directly on the EDP. Therefore, every performance objective is realized as the exceedance probability of exceeding a specified performance level. Following this concept, for every performance level we calculate its mean annual frequency (MAF) of exccedance (v_{LS}). The calculation of the MAF can be derived using the total probability theorem:

$$v_{LS}(edp \le EDP) = \int_{0}^{+\infty} P(edp \le EDP \mid IM = im) \left| \frac{dv(IM)}{dIM} \right| \mathrm{d}IM \tag{7}$$

$P(edp \le EDP \mid IM = im)$ is the probability of limit-state being exceeded, termed also as fragility or vulnerability function, and $\left|\mathrm{d}v(IM)/\mathrm{d}IM\right|$ is the slope of the hazard curve. The absolute value is used to prevent from the negative value of the slope of the hazard curve. Equation 7 convolves the ground motion uncertainty, given through the hazard curve of the site, with uncertainties regarding the structural performance represented by the building's fragility curve.

The equation is calculated numerically since the analytical integration is not always possible. There are two ways to calculate the MAF [13]. The first way is calculating the probability that the demand exceeds the capacity of the structure, called the direct or EDP-based method, or by alternatively using the indirect or IM-based approach. The IM-based approach refers to calculating the probability that the IM will be above the random IM capacity of the structure. In this work the latter method is used, where:

$$P(edp \leq EDP \mid IM = im) = P(IM_C < IM \mid IM = im) \quad (8)$$

The mean annual frequency of exceedance of a limit-state is estimated using the statistics of the responses calculated with the aid of SPO2IDA. SPO2IDA gives an estimate of the mean value and the standard deviation of the response and may be used to calculate equation 7, for given EDP value. This is based on the assumption that the IM values are lognormally distributed. The probability of exceeding the IM capacity of the structure is thus calculated and multiplied with the slope of the hazard curve using equation 8. If $\ln(\hat{\theta}_{\max})$ and $\hat{\delta}$ are the logarithmic mean and the standard deviation of $\hat{\theta}_{\max}$ for given intensity $S_a(T_1,5\%)$, $\hat{\delta}$ is calculated as $\hat{\delta} = 0.5 \cdot \left(\ln S_a^{84\%} - \ln S_a^{50\%}\right)$ [14].

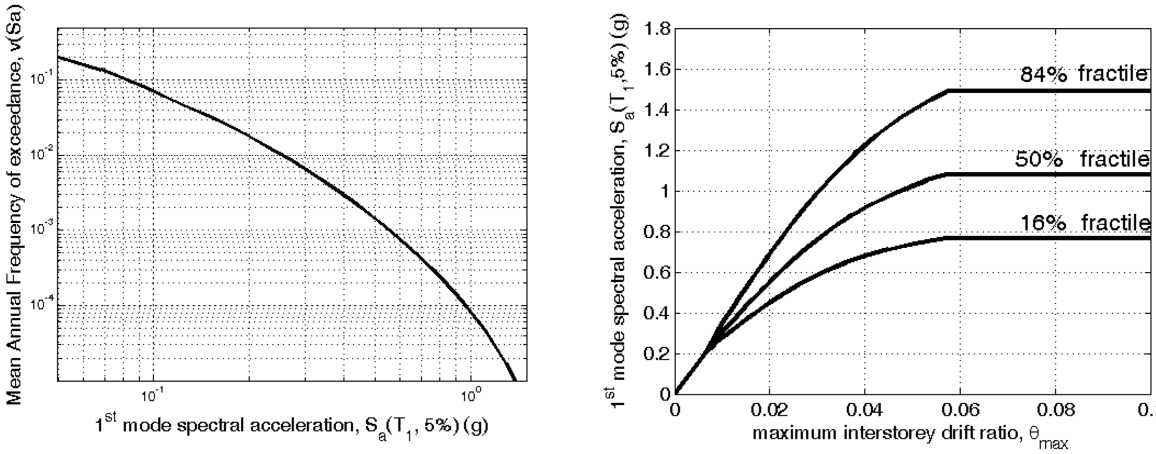


Figure 2 (a) Hazard curve for T_1= 0.93 sec, and (b) median SPO2IDA curve and its 16th and 84th fractiles.

The seismic hazard at a site can be obtained through probabilistic seismic hazard analysis (PSHA) and is represented by a hazard curve (Figure 2a). The performance levels correspond to exceedance probabilities equal to 50%, 10% and 2% in 50 years (briefly denoted hereafter as 50/50,10/50 and 2/50). For example, the IO level implies very light damage with minor local yielding and negligible residual drifts within a period of 50 years corresponding to a level of 50% probability of exceedance. Using the first-mode spectral acceleration of the structure and its period it is possible to obtain the mean annual frequency of exceedance of the ground motion *v(IM)*.

The probabilistic constraints are applied on the annual rate of the drift value being exceeded for every limit-state considered. In particular the rates used for the 50/50, 10/50 and 2/50 hazard levels are related to the return period of the limit-state being exceeded with the relationship $\tau_{LS} = 1/\nu_{LS}$. The corresponding return periods are 72, 475, 2475 years respectively. This leads to the following probabilistic constraints:

$$\begin{aligned} &\tau_{IO} \geq 72\text{yrs} \\ &\tau_{LS} \geq 475\text{yrs} \\ &\tau_{CP} \geq 2474\text{yrs} \end{aligned} \quad (9)$$

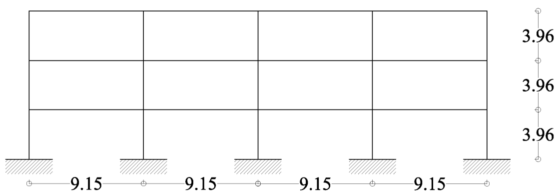
The penalty adopted for the i-th limit state of the reliability-based formulation of the optimization problem is:

$$p_\tau = \frac{\tau^{\mathrm{i}}_{\mathrm{lim}} + \tau^{\mathrm{i}}}{\tau^{\mathrm{i}}_{\mathrm{lim}}} \tag{10}$$

where $\tau^{\mathrm{i}}_{\mathrm{lim}}$ is the return period of the i-th limit state set by the codes and τ^{i} is the return period of the design given by the optimization algorithm.

4 NUMERICAL RESULTS

The proposed methodology is demonstrated on a three-storey steel moment-resisting frame (SMRF) (Figure 3(a)). The frame has been designed for a Los Angeles site according to the 1997 NEHRP (National Earthquake Hazard Reduction Program) provisions. All analyses were performed on the OpenSees platform [15]. The modulus of elasticity was assumed equal to 200GPa and the yield stress f_y=235MPa. All sections are W-shaped, taken from the tables of the American Institute of Steel and Construction (AISC). The frame is assumed to have rigid connections and fixed supports. The permanent load is taken as G=5KN/m^2 and the live load is considered equal to Q=2KN/m^2. The EDP adopted is the maximum interstorey drift, $\theta_{\max}$, and the thresholds were 0.6, 1.5, and 3% for IO, LS and CP levels, respectively. This building is a first-mode dominated structure.



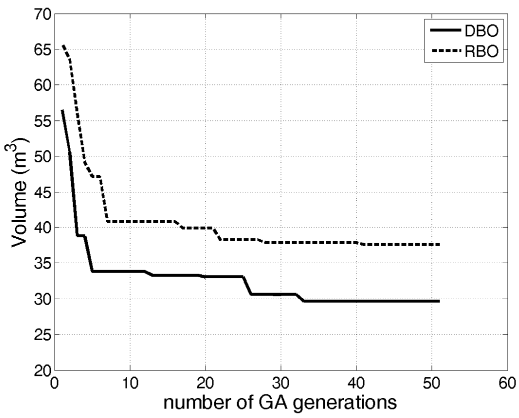


Figure 3 (a) Three-storey, steel moment resisting frame, and (b) optimization history of the three-storey frame using the deterministic-based and the reliability-based procedure.

The optimum designs obtained are shown in Table 1. As can be seen, the MAF of exceedance of the optimum design for the reliability-based optimization case satisfies the probabilistic constraints. The MAF of the optimal design using deterministic-based criteria only is given as well additionally. Figure 3(b) shows the best objective function value as the generations converge to the optimum design. It can be seen that the optimum weight reduces from 58m^3 to 29.64m^3 in the case of deterministic-based design, while for the reliability-based case, the optimum weight reduces from 65m^3 to 37.55m^3.

Case study	Volume (m^3)	Optimal design	MAF
DBO	29.64	W33×201, W27×94,W21×50, W14×30, W14×38	IO 1×10^{-2} LS 2.8×10^{-2} CP 4.7×10^{-4}
RBO	37.55	W33×263, W27×94,W21×68,	IO 1.7×10^{-3} LS 0.9×10^{-3}

W14×48,W14×26	CP	0.1×10^{-3}

Table 1. Optimal design results for the two buildings used.

Figure 4 compares the capacity curves of IDA and SPO2IDA for the optimal design of the three storey considered using the reliability-based design procedure. For the reliability-based design of the three-storey SMRF (Fig. 4) the two curves seem to be in good agreement in the elastic range. The capacity is overestimated for limit states between the elastic range and until $\hat{\theta}_{\max}$ =0.054 while beyond this value as the frame approaches collapse, SPO2IDA underestimates the capacity. These discrepancies introduce a small error in curve calculations, which is sufficient for an automatic design algorithm.

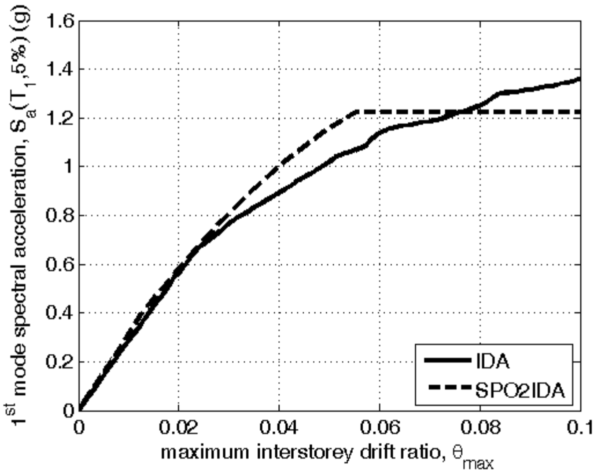


Figure 4 Median IDA curve and its SPO2IDA approximation. The curves refer to the optimal design of the three storey frame obtained by the reliability-based procedure.

5 CONCLUSIONS

A new seismic design procedure for steel moment frames has been developed. The proposed design procedure is expressed with the aid of deterministic and/or probabilistic design criteria. Both types of criteria can be imposed within the performance-based design concept as suggested by the FEMA guidelines with the latter enabling the engineer to define the return periods of preset performance levels. In this way, a common language can be used within the engineers and stakeholders during the building design procedure. The proposed algorithm uses approximate performance-estimation methods and in particular the SPO2IDA method. We have shown that the implementation of structural design code checks is possible and designs that meet the code provisions can be obtained in a straightforward manner. The mean annual frequencies of the limit-states considered are compared to preset values in order to decide whether each candidate design is acceptable. A Genetic Algorithm (GA) was implemented for the solution of the design problem leading to efficient optimal solutions through an iterative procedure. The results obtained reveal the efficiency of the proposed approach for first-mode dominated structures reducing considerably the computing time.

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