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PROBABILISTIC EVALUATION OF SEISMIC PERFORMANCE FOR A STEEL MOMENT FRAME USING DAMAGE INDICES

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Abstract. *Reliable predictions of seismic structural damages are of essential importance in earthquake engineering. In order to make proper and effective decisions in post-earthquake situations, it is necessary to have realistic estimations of the extent of seismic damages occurred in individual structures. For this purpose, structural damage indices can be very useful. Defined in a variety of forms, they can serve as strong analytical tools needed to calculate different aspects of structural damages caused by earthquakes.*

This study aims to provide a comparison among some of the well-known analytical damage indices, within the scope of seismic performance evaluations, and using a probabilistic framework.

A sample 5-storey special moment-resisting frame is modeled using OpenSees. Damping, seismic mass, yield strength and ultimate strength of steel are described as probabilistic variables, and random realizations of the structure are generated using Latin Hypercube Sampling (LHS) method. Sample frames are then analyzed through Extended IDA. Increasing levels of damage, due to increasing levels of strong ground motion intensity, are described with the selected set of seismic damage indices.

Considering the results, each damage index displays a different pattern for IDA curves based on the response parameters used in it. Seismic performance levels, as defined in some seismic design criteria, are correlated to the order of damage indices' values. Also, statistical dispersion in a set of damage values corresponding to a particular performance level is different for each index, indicating different levels of sensitivity to epistemic uncertainties among damage indices.

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1 INTRODUCTION

The essential scope of seismic performance evaluations is to restrict the extent of damages caused by probable seismic events. However, complicated and uncertain nature of seismic loads and non-existence of perfect knowledge about structural behavior makes it very difficult to have a reliable estimation of seismic damages.

Quantification of some of the seismic damage characteristics such as type, severity and distribution can present an illustrative depiction of post-earthquake status for structures. Such information can be very helpful in seismic retrofit decision making, disaster planning and post-earthquake assessments [1]. In this regard, engineering practices need analytical methods which can give realistic predictions of damage characteristics.

In recent decades, many researchers have focused on providing methods that can give a suitable description of damage state in a loaded structure. 'Damage Indices', or in mathematical terminology, 'Damage Functionals' are functional relationships that synthetically represent the damage imposed on structure with a real number [2]. In a normalized form, output of these functions is a number between 0 and 1, which 0 represents no damage and 1 represents rupture or complete failure.

Comparative studies exist which can illustrate how different damage indices correlate with each other in different levels of plastic deformations. In case of steel structures, Castiglioni and Pucinotti [3] introduced failure criteria and a model of damage accumulation for the welded beam-to-column connections and compared it with some of the most common damage models available in the technical literature. Arjomandi et al. [4] considered the correlation among FEMA-356 [5] performance levels and the values of different damage indices for a set of steel frames. They also suggested some polynomial equations that can be used to easily estimate the value of each damage index.

Kamaris et al. [6] proposed a new damage index for planar steel moment resisting frames (MRF) and compared it with five widely used damage indices existing in the literature. Their study displays the correlation among values of these damage indices for increasing levels of ground motion intensity. Also, Kamaris et al. [7] presented simple empirical expressions to estimate maximum seismic damage on the basis of the five selected damage indices for planar regular moment resisting and x-braced steel frames.

As for the present study, two main goals exist: first, it is intended to use various damage indices as the Damage Measure (DM)[1] in performance evaluations. In definition, "a DM is an observable quantity that is part of, or can be deduced from, the output of the corresponding non-linear dynamic analysis" [8]. Incremental Dynamic Analysis (IDA) has the ability to utilize various damage indices as DM [8]. Nonetheless, with a review on literature, it can be seen that the Maximum Interstory Drift Ratio (IDR) has been the main choice for this parameter so far. Hence, it seems very useful to study the advantages of using powerful damage indices for the purpose of producing IDA curves.

The second goal of this paper is to consider the effects of structural modelling uncertainties on seismic damage indices. By reviewing technical literature, it can be seen that this subject has not been of particular interest yet. Every damage index uses certain structural response parameters, and each of these parameters have a degree of sensitivity to epistemic uncertainties which deterministic procedures cannot reflect. Therefore, evaluation of damage indices in a probabilistic framework can generate numerous meaningful results.

[1] Defined as Engineering Demand Parameter (EDP), according to the current Pacific Earthquake Engineering Research Center terminology

2 CATEGORIZATION OF DAMAGE INDICES

Comprehensive studies on classification of damage indices, plus detailed discussions about advantages and disadvantages of each one can be found in the literature [1, 9, 10]. For the sake of brevity, only a concise description about damage indices will be provided prior to introduction of the ones used for this study. As it can be seen in literature, there are two major approaches toward classifying quantitative damage indices:

In one approach, they are categorized as Cumulative or Non-cumulative, meaning whether they are capable of taking into account the accumulation of damage in repeated load reversals. This distinction can become important for loadings with a dynamic nature such as earthquake. Non-cumulative indices are mostly governed by the maximum deformation occurred in loading history. Some of these indices can account, to some degree, for effects of cyclic loading by means of including strength and stiffness degradation [11, 12].

Although non-cumulative indices may be incapable of displaying effects of some important factors in dynamic loadings (like duration, frequency content or distribution of inelastic cycles), they are still widely used in structural engineering applications because of simplicity in definition and application [1].

Cumulative indices, on the other hand, contain a specific component that makes it possible to consider the cumulative aspects of damage .This component can be based on deformation [13], hysteretic energy [14], or low-cycle fatigue formulations [15]. Some combinatory indices are also proposed. For example, combination of excessive deformations and dissipated energy [16], or a combination of strength damage and deformation damage divided by total area between the monotonic load deformation curve and the fatigue failure envelope [17].

In another approach, damage indices are categorized as Local or Global. Local indices calculate damage only on a local level (individual members), and in order to calculate the overall damage of structure it is needed to use weighting methods [17, 18]. Global indices are developed to calculate damage directly on the global level [9, 19]. These indices give a representative value for the overall damage to structure (or a storey) and detailed information about the distribution of damage between different elements will not be available.

3 DESCRIPTION OF DAMAGES IN A PROBABLISTIC FRAMEWORK

According to FEMA-350 [20], Inter-storey Drift angle is the preferred parameter to describe the performance of steel MRF. As stated in these criteria, this parameter "(1) seems to be stable with regard to prediction of frame performance, (2) is closely related to plastic rotation angle, (3) is less ambiguous with regard to definition, and (4) is a quantity that is easily determined from the results of standard frame analyses using either linear or nonlinear methods."

However, due to being a non-cumulative parameter, it is incapable of predicting some aspects of seismic damages. The main advantage of using more sophisticated damage models is to overcome this very problem. Advanced damage indices may be more difficult to be calculated, and may even have rather ambiguous definitions. Nevertheless, when it comes to loading histories such as earthquakes, they can provide us with a much better understanding of how damages propagate through structures during loading.

Another perspective that can be added to this problem is quantification of structural modelling uncertainties. Instead of usual deterministic approaches, evaluation of structural capacity against seismic demand can be performed in a probabilistic framework. Each of the parameters comprising mathematical model of a structure are subjected to some level of uncertainty, and these uncertainties are able to change the results of performance evaluations [21, 22].

In this paper, both sources of epistemic and aleatory uncertainties are incorporated into the results. For this purpose, the structure is analysed in two forms: deterministic and probabilistic. In the first phase, for a selected set of accelerograms, structures are analysed through IDA with their parameters set to central values, which will be named as *Base Case* hereafter. Results of this phase only include uncertainties due to the record to record effect.

In the next phase, damping, mass, yield strength and ultimate strength of steel are assumed as probabilistic variables, and a sufficient number of different realizations of structural model are generated. Then, every single realization of the structure is subjected to IDA for selected records. Results of this phase, which will be named as *Uncertain Case* hereafter, include effects of both aleatory and epistemic uncertainties.

In order to optimize the procedure of generating random realizations of the structure, Latin Hypercube Sampling (LHS) method [23] has been used here. This technique uses a constrained sampling scheme instead of random sampling utilized by direct Monte Carlo method, and consequently will need significantly fewer simulations to cover desired probability space [24, 25]. This property can become very helpful to decrease the high computational costs usually associated with uncertainty studies.

As for the selection of intensity measure (IM) parameter, it has been suggested that even in the presence of mass and stiffness uncertainties, the use of spectral acceleration corresponding to fundamental period of base structure as IM parameter can be considered as an appropriate solution [21].

After gathering pair values of intensity and demand from multi-record IDA curves, statistical characteristics of their distribution can be calculated for any desired performance level. Central values (median or mean) represent structural capacity at performance levels, and dispersion values (standard deviation) represent the extent of uncertainty effects.

4 DAMAGE INDICES USED FOR THIS STUDY

Five damage indices are selected for this study: Ductility Ratio [26, 2], Modified Flexural Damage Ratio (MFDR) [12], Park and Ang [16], Krawinkler and Zohrei [15], and Final Softening [27]. Selection of these indices were based on two criteria: Firstly, they are well known amongst researchers and have been used in a numerous studies. Secondly, they are formulated with diverse theoretical bases which allows us to observe how different theories lead to different estimations of damage.

4.1 Ductility Ratio

This local damage model, which was first introduced in 1988 [26], simply defines damage as the ratio between the maximum deformations occurred and a predefined value of deformation corresponding to failure:

$$\mu = \frac{\delta_m - \delta_y}{\delta_u - \delta_y} \tag{1}$$

δ_m, δ_y and δ_u are the maximum, yield and ultimate values of damage parameter respectively. Various options such as curvature, rotation or displacement can be used as the damage parameter of this model. In a normalized form, this index will be calculated as:

$$D_{DR}=\frac{\delta_m-\delta_y}{\delta_u-\delta_y}=\frac{\dfrac{\delta_m}{\delta_y}-\dfrac{\delta_y}{\delta_y}}{\dfrac{\delta_u}{\delta_y}-\dfrac{\delta_y}{\delta_y}}=\frac{\mu_m-1}{\mu_u-1} \tag{2}$$

δ_u can be defined by means of a monotonic static loading. From seismic analysis of structural capacity to development of nonlinear response spectra of structures, this index has been used in a variety of applications [9].

4.2 Modified Flexural Damage Ratio

Roufaiel and Meyer proposed this index in 1987 for concrete structures, although it has been used for steel structures as well [7]. In this model, damage is described as the ratio between the secant stiffness of a member at its onset of failure, and the minimum secant stiffness reached so far in the moment-curvature relationship. The value of damage is calculated independently for both negative and positive directions of loading:

$$MFDR^{+}=\frac{\dfrac{\phi_x^{+}}{M_x^{+}}-\dfrac{\phi_y^{+}}{M_y^{+}}}{\dfrac{\phi_m^{+}}{M_m^{+}}-\dfrac{\phi_y^{+}}{M_y^{+}}} \tag{3}$$

$$MFDR^{-}=\frac{\dfrac{\phi_x^{-}}{M_x^{-}}-\dfrac{\phi_y^{-}}{M_y^{-}}}{\dfrac{\phi_m^{-}}{M_m^{-}}-\dfrac{\phi_y^{-}}{M_y^{-}}} \tag{4}$$

$$D_{RM}=Max\{MFDR^{+},MFDR^{-}\} \tag{5}$$

In Equations 3 to 5, ϕ_m/M_m is the inverse of secant stiffness at the onset of failure, ϕ_x/M_x is the inverse of minimum secant stiffness reached so far, and ϕ_y/M_y corresponds to initial elastic stiffness. Obviously, if a section has symmetrical moment-curvature diagram in negative and positive loadings, the value of $MFDR^-$ will be equal to $MFDR^+$.

4.3 Park and Ang

This index was introduced in 1983 and was originally developed for concrete structures. It can be said that this index is the most famous one with the greatest number of citations from studies concerning both concrete and steel structures [7, 4]. In this index, damage is modelled as the linear combination of ductility (excessive deformations) and dissipated energy in loading cycles which takes the following form:

$$D_{PA}=\frac{\delta_m}{\delta_u}+\frac{\beta}{Q_y\delta_u}\int dE \tag{6}$$

In Equation 6, δ_m is the maximum deformation in loading history, δ_u is the ultimate deformation under monotonic loading, Q_y and Q_u are yield and ultimate strength respectively, dE is the increasing hysteretic dissipated energy, and β is a non-negative coefficient that is determined through calibration against experimental results. According to experimental studies conducted on H-shaped steel cantilevers, a mean value of 0.025 can be considered for the β coefficient [28].

4.4 Krawinkler and Zohrei

This index was introduced in 1983, based on the low-cycle fatigue theory. The Coffin-Manson relationship is used to describe the relationship between the required number of cycles to failure and the amplitude of deformations, and the linear damage accumulation law of Miner [29] is applied.

This index is capable of considering two separate modes of failure: local buckling of elements, and fracture at weldments due to propagation of cracks. For the case of damage caused by local buckling, deterioration in three modes are evaluated which consist of stiffness, strength and energy dissipation:

$$D_{KZ} = \frac{d}{x} \tag{7}$$

$$d = \sum \Delta d_i \tag{8}$$

$$\Delta d_i = C(\Delta \delta_{pi})^c \tag{9}$$

In Equations 7 to 9, D_{KZ} is the calculated value of damage index, d is the cumulative value of damages occurred in different loading cycles (Δd_i), x is a predefined value of damage that is assumed as the ultimate capacity of element, and $\Delta\delta_{pi}$ is the range of plastic deformations in a specific loading cycle. Finally, C and c are scalars that can be defined based on graphs provided for this purpose [15], depending on the type of steel, the cross section of element and also the deteriorating mode under consideration (stiffness, strength or energy dissipation).

4.5 Final Softening

Dipasquale and Cakmack introduced three models of damage based on evaluation of fundamental period of structures [19], all of which are categorized as global damage indices. Final softening is an index which calculates damage only based on variation in the fundamental period of a structure, before and after seismic loading:

$$\delta_f = 1 - \frac{(T_0)^2_{initial}}{(T_0)^2_{final}} \tag{10}$$

In Equation 10, T_0 displays fundamental period of the structure. In practical applications, final period is determined via in-situ vibration tests on structures [27]. Moreover, in analytical applications, this index can be simply calculated by determination of structure's fundamental period in the beginning and at the end of seismic loading.

4.6 Weighted average indices

Among the methods suggested in literature which convert damage values of individual members to a global value, two of them are most favored. In one these approaches [18], local indices are weighted based on their local energy absorptions:

$$D_{story} = \frac{\sum E_i D_i}{\sum E_i} \tag{11}$$

In Equation 11, D_i is the local damage for a particular member and E_i is the energy absorbed by it. The other method [17], has a more generalized form which considers an exponential form for local indices:

$$D_{story} = \frac{\sum w_i D_i^{b+1}}{\sum w_i D_i^{b}} \quad (12)$$

In Equation 12, D_i is the damage incurred at a specific structural member, and w_i and b are arbitrary coefficients. In the simplest form, w_i and b can be taken equal to 1, which means structural elements have equal proportions of the overall damage regardless of their location or their role in the structural system. In this paper, Equation 12 has been utilized to calculate the weighted average of indices with both w_i and b set to 1.

5 STRUCTURAL SYSTEMS AND NUMERICAL MODELLING

5.1 General properties of the frame

The structure under consideration is a 5-storey 3-bay steel MRF, in which height of stories is 3.20 m and width of bays is 5.0 m. According to criteria and definitions of Iranian Steel Design Code [30] it can be categorized as a special moment resisting frame (SMRF). It is worth mentioning that provisions given in the Iranian national code are very similar to those of AISC [31] and FEMA-350 [20]. Based on modal analysis of the frame, this structure has a fundamental period equal to 1.12 seconds.

5.2 Important details of analytical modelling

OpenSees [32] is utilized to create mathematical model of the moment frame and also to perform nonlinear analyses. Nonlinear beam-column elements are used to constitute the frame, and fiber section method is implemented to incorporate spread plasticity into nonlinear behaviour of elements. This method forms a member as a group of fibres, each of which can have a uniaxial force-deformation behaviour defined by the user.

Steel02 material from the software library is selected to describe the hysteretic and monotonic behaviours of steel, and fatigue material is utilized to wrap Steel02 with a limit value on deformations (i.e. for deformations exceeding the pre-defined value, failure will occur).

Although involving some features like panel zones would make the numerical model more realistic, in order not to enter strength and ductility of panel zones into the probabilistic framework, here it is decided to use an elastic centreline model for beam-column connections. The corotational method is used for geometric nonlinearities of frames [33]. Also, Rayleigh damping model is used to form classic damping matrice of the structure.

The final point in mathematical modeling is about efficient seismic mass of the frame. Here, this parameter is calculated from dead loads plus 20% of live loads. In formation of the mass matrices, the assumption of concentrated nodal mass is used. This assumption means that elements are weightless in their length, but an amount of mass is assigned to each node considering adjacent lengths of loading, leading to formation of a diagonal mass matrice.

6 QUANTIFICATION OF UNCERTAINTIES

6.1 Parameters of the structural model

When using the LHS method to generate a pseudo-random set of numbers for desired parameters, it is needed to have specific statistical target distributions describing probability distribution functions (PDF) of the parameters, as well as their correlation matrix.

In order to introduce the PDF of ratio of equivalent viscous damping (ξ), here the work of Porter et al. [34] is used, in which the authors have compiled the results of some researches aimed at estimation of ξ for different kinds of structures. Porter et al. concluded that a reasonable value for coefficient of variation (C.O.V) for this parameter should be between 0.3 and 0.4. Based on this finding, and some other studies [35], here it is chosen to assign a lognormal distribution with median of 0.05 and C.O.V of 0.4 for parameter ξ.

Structural mass acting during earthquake is formed from dead loads, in addition to a percent of live loads imposed on the structure. Although live loads used in the design of buildings are exposed to many uncertainties, mainly because they constitute a relatively less important portion of efficient seismic mass, here it is decided to treat them deterministically.

Based on the results of some other studies, Ellingwood et al. [36] concluded that a proper way to describe dead loads as a probabilistic variable is to assume a normal distribution with a mean value equal to dead loads used in the design procedure and a C.O.V equal to 0.1. The same suggestion has been employed in this paper.

PDF of yield strength (F_y) and ultimate strength (F_u) of steel are assumed as introduced in the report No.177 of John A. Blume earthquake engineering center [37]. Based on data gathered from tensile strength tests conducted on flange coupons, this report has presented statistical properties of PDFs for F_y and F_u as shown in the Table 1 (σ: standard deviation, ρ: correlation coefficient):

Mean F_y (Mpa)	σ_{Fy} (Mpa)	Mean F_u (Mpa)	σ_{Fu} (Mpa)	$\rho_{Fu,Fy}$
310.3	35.8	455.7	29.6	0.851

Table 1: Statistics of material yield strength from flange coupon tests [37]

It should be mentioned that other than F_y and F_u, no correlation has been considered for any other pair of parameters (i.e. mechanical strengths of steel, damping and mass are assumed completely independent from each other).

Based on the statistical target distributions and the correlation matrix described above, a set of 75 random realizations are generated for each structure. Compared to some other studies in this area [24, 25], this number of realizations seems to be sufficient.

6.2 Strong ground motions

Strong ground motions caused by earthquakes have a highly complicated nature. Based on characteristics like frequency content, peak acceleration, peak velocity and energy content, each record may have a specific influence on the structure being considered. Therefore, a wise strategy to contain record specific effects is to choose a set of records that have a wide range of variation in their intrinsic characteristics.

From another aspect, demands achieved from nonlinear time history analyses should be somehow consistent to the seismicity of area under study. Hence, selected accelerograms must be evaluated from a hazard analysis point of view as well.

Following these two criteria, a set of 12 records has been selected, which based on NEHRP [38] classification correspond to the soil category C and are all free of near-field effects. General properties of these records are presented in Table 2.

No.	Event	Year	Station	Moment Magnitude	Distance (km)
1	Cape Mendocino	1992	Eureka	7.01	42
2	Duzce	1999	Lamont 1062	7.14	10.2
3	Imperial Valley	1979	Superstition	6.53	25.23
4	Kern County	1952	Taft	7.36	38.89
5	Kobe	1995	Nishi-Akashi	6.9	8.12
6	Kocaeli	1999	Arcelik	7.51	13.52
7	Loma Prieta	1989	Gilroy	6.93	9.96
8	Loma Prieta	1989	Anderson Dam	6.93	20.26
9	Manjil	1990	Abbar	7.37	12.97
10	Northridge	1994	L.A Baldwin Hills	6.69	29.88
11	Northridge	1994	Obregon Park	6.69	37.36
12	Victoria	1980	Cerro Prieto	6.33	14.37

Table 2: Earthquake records used for nonlinear time history analyses.

Also, to see how these records match with seismicity of the hypothetical region of the structure, the linear acceleration spectra of records against 475 years uniform hazard spectra (UHS) of the region are depicted in Figure 1. The UHS curve is extracted from seismic hazard analysis project established by President Deputy Strategic Planning and Control [39]. As it can be seen from Figure 1, mean spectral acceleration of the selected records is well matched to the UHS of area. Hence, it can be said that these records, in an average sense, are representative of seismicity of the area.

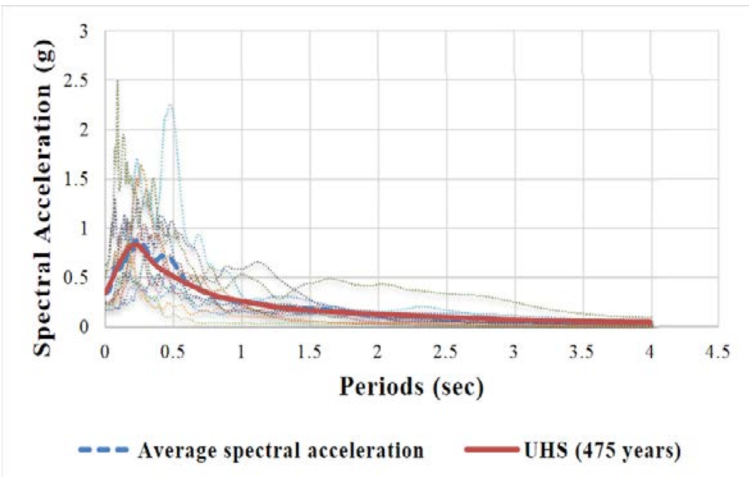

Figure 1: Spectral acceleration of selected records, and their average against 475 years UHS of region.

7 RESULTS

7.1 Capacity curves based on various damage indices

For the first part of results, summarized IDA curves representing 16th, 50th and 84th percentiles of structural response are depicted in Figures 2 to 7. In each figure, three curves correspond to the *Base Case*, and the other three correspond to 75 random realizations named as the *Uncertain Case*.

An essential item regarding post-processing of the data gained by IDA analyses is the upper-limit set on the damage values for each single-IDA curve, i.e. where the flat-line of each curve is determined to start. In this paper, it is simply assumed that when the weighted average of damage values at a storey becomes equal to 1.0, the structure reaches to a globally unstable status. When interpreting the results shown in the following figures, it should be noticed that with selection of a different upper-limit, ultimate capacities will also change leading to considerable differences in percentile curves.

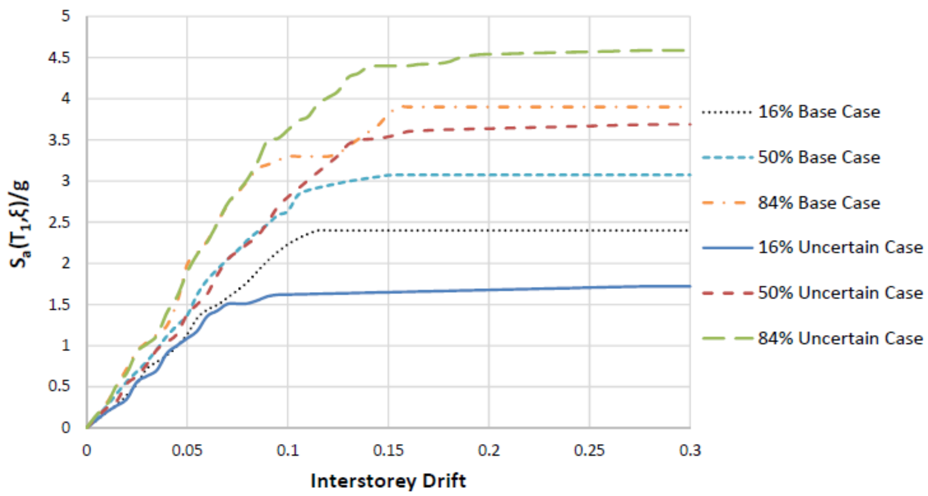

Figure 2: Summarized IDA curves based on Interstory Drift Ratio.

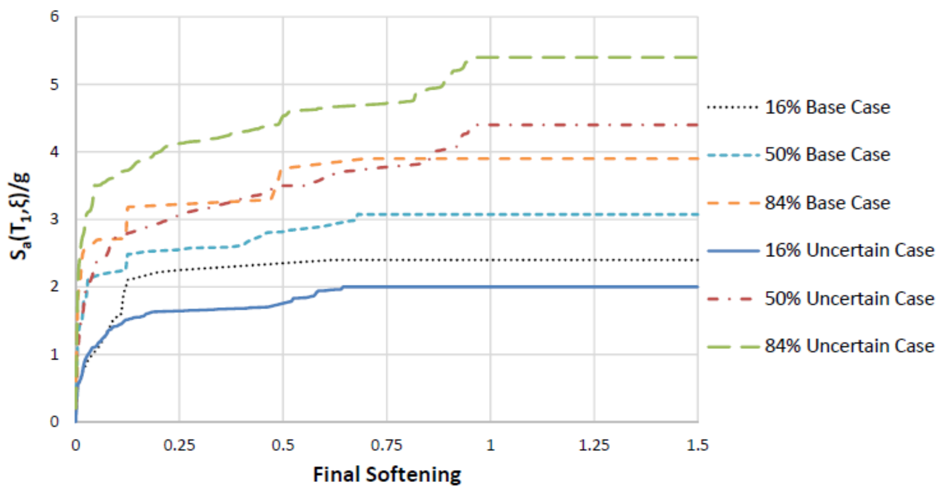

Figure 3: Summarized IDA curves based on Final Softening damage index.

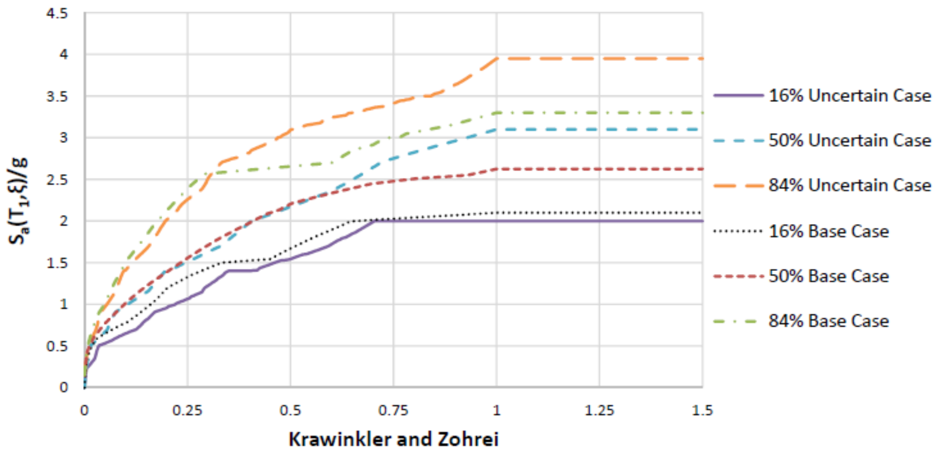

Figure 4: Summarized IDA curves based on Krawinkler and Zohrei damage index.

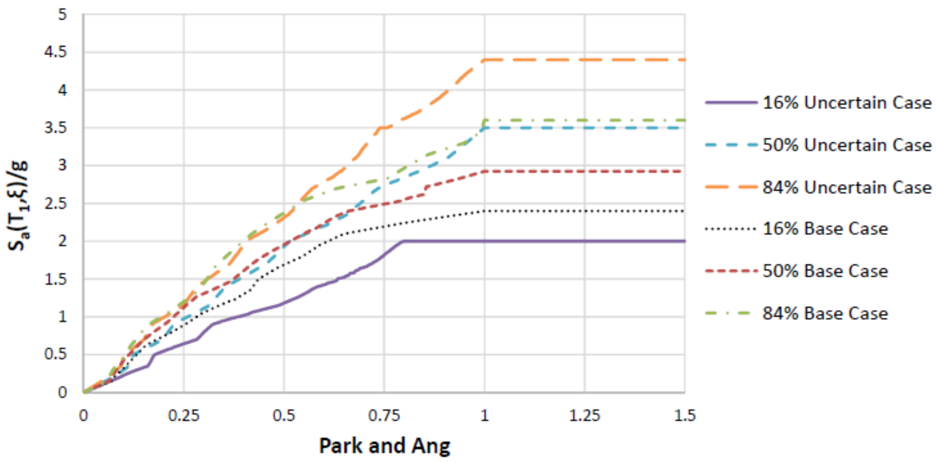

Figure 5: Summarized IDA curves based on Park and Ang damage index.

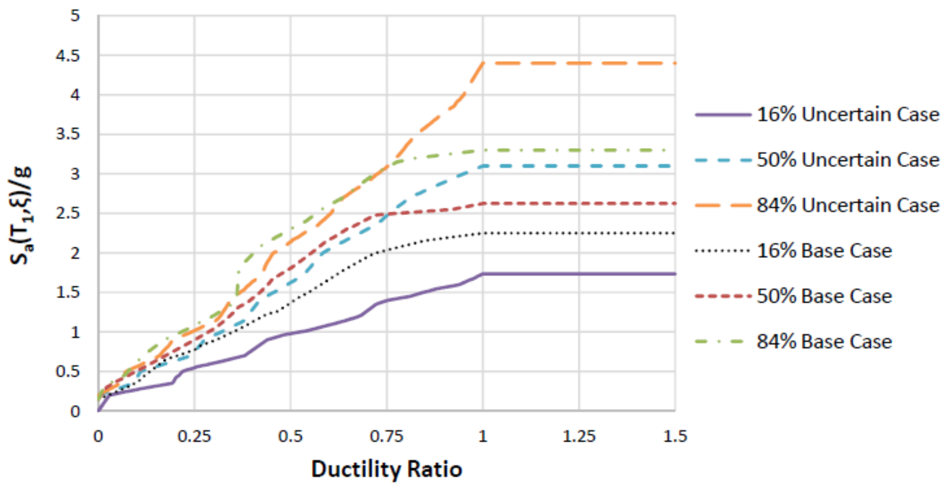

Figure 6: Summarized IDA curves based on Ductility Ratio damage index.

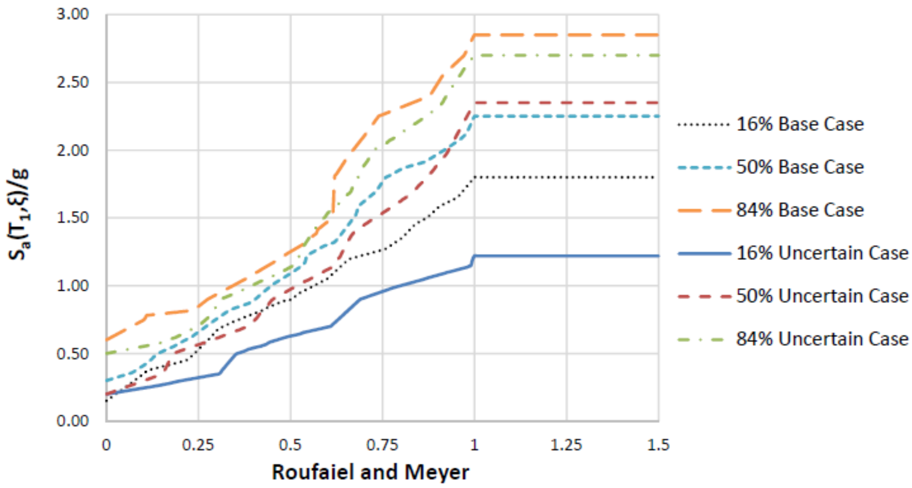

Figure 7: Summarized IDA curves based on Roufaiel and Meyer damage index.

A point common among all of the indices is that the median of structural capacities in uncertain realizations differ from that of the base structure. In better words, as discussed in details in [21], the median-parameter model does not necessarily generate the median seismic response.

As it can be seen from the Figures 2 to 7, each damage index shows a distinct pattern in its IDA curves. The trend in which slopes of the curves change from linear state to flat-line can be indicative of how a damage measure describes the propagation of nonlinearity in the frame at increasing levels of seismic intensity. In the following, each of the patterns observed will be discussed separately for more details:

- D_{DR}: This non-cumulative index shows a pattern which is similar to IDR, meaning that slopes of the percentile curves gradually decrease from the initial slope to zero. However, percentile curves of this index are generally below the corresponding ones from IDR.

- D_{RM}: This damage measure does not show any considerable decrease in stiffness or strength of the structure right before the flat-line is reached. Capacities calculated by this index are the lowest among all. In addition, compared to other local indices, the threshold value of this index clearly affects the start of each curve. For example, at 84th percentile of both Uncertain and Base Cases, damage will remain zero until the seismic intensity becomes equal or more than 0.5g.

- D_{PA}: Similar to D_{DR} and IDR, this index shows a gradual transition from linear stage to global instability. Structural capacities estimated by this index have a better correlation to corresponding ones from IDR. It is also noteworthy that damages are always greater than zero, even when the seismic intensity has very small values. This matter is due to the fact that D_{PA} does not have a threshold value, i.e. for any value of seismic forces there will be a positive value of δ_m (Equation 6), which makes the damage measure greater than zero.

- D_{KZ}: compared to other local indices, these curves have steeper slopes in the beginning, and their slopes reduce at a higher rate to reach flat-line. In addition, except for 16th percentile at Uncertain Case, the percentile curves of this index are below the ones from Inter-storey Drift.

- δ_f: As the only global damage index used in this study, it shows a pattern which is clearly distinct in several ways. Please note that the initial slopes are very steep, and in some percentiles, an intensity as high as 2.0g is needed to initiate the damage. However, after only a limited value of damage index, strength and stiffness rapidly start to decrease. In most percentiles, the curves reach their flat-line before half of the ultimate value has occurred.

7.2 Description of FEMA-350 performance levels based on damage indices

According to definitions existing in FEMA-350 [20], performance limit states can be defined for steel MRF. Particularly, three limit states with titles of immediate occupancy (IO), collapse prevention (CP) and global instability (GI) are of prime interest. Each of these three corresponds to a certain level of development of nonlinearity in structural behaviour, indicating of how seismic damages have affected performance of the structure.

As can be expected, limit states are assigned with specific values of IDR [20], and these values will be the criteria of achieving the desired performance. In case of special moment resisting frames (SMRF), IO will be exceeded at IDR>2%; and CP will be reached when IDR=10% or when the slope of the IDA curve is reduced to 20% of its initial value, whichever occurs first.

Obviously, it will be a strenuous task to redefine these criteria based on a particular damage index, because such a procedure needs calibration of damage models against a large set of data gained from experiments and real-world observations. Nonetheless, correlating values of damage indices with the criteria presented in FEMA-350 [20] utilizing numerical simulations can provide us with a general idea of how various damage indices describe the structural status at different performance levels.

In this paper, the basis of correlating IDR to other damage indices is the equivalency of structural capacities. In better words, the pair values of IDR and a specific damage index are assumed to be counterparts when the structural capacity is equal between them. Based on this assumption, the average values of $S_a(T_1,\xi=0.05)$ at 2% and 10% of IDR are calculated, and then the quantity of each damage index corresponding to these average structural capacities are extracted from their relevant multi-IDA performance curves. The results of this procedure are tabulated in the following:

Damage Measure	Base Case IO Level	Uncertain Case IO Level	Base Case CP Level	Uncertain Case CP Level
IDR	0.02	0.02	0.10	0.10
D_{DR}	0.12	0.16	0.98	0.87
D_{RM}	0.14	0.25	1.0	1.0
D_{PA}	0.13	0.15	0.85	0.77
D_{KZ}	0.02	0.03	1.0	0.79
δ_f	0.0	0.0	0.41	0.12

Table 3: Correlation among different damage indices at IO and CP performance levels.

Considering the results in Table 3, it can be seen that the amounts representative of a performance level can significantly vary among damage indices. For example, at the IO level, while δ_f and D_{KZ} have negligible values, the other three estimate damages greater than 0.10. In addition, damage approximations can have considerable variations between the Base Case and the Uncertain Case. This variation is most important in the case of δ_f, where epistemic uncertainties reduce the representative damage value by 70 percent.

It is also noteworthy that some of these indices represent CP level with a damage quantity equal (or very close) to 1.0. It is reminded that one of our main assumptions was equivalency between global instability of the structure and a weighted average of 1.0 in the story level. As the result of this assumption, the procedure implemented in this paper cannot differentiate between CP and GI limit states for some of the local indices.

To shed some light on this matter, it is necessary to elaborate on the weighting method used herein. Referring to Equation 12, it can be seen that contribution of beams and columns to the damage amount of a story is assumed identical. However, it is well known that the requirement of "strong column-weak beam" in the design of SMRF necessitates beams to undergo a lot of more plastic deformations compared to columns up to the point of structural collapse. Therefore, the weighted average of damages in a story can reach the ultimate value while only a relatively small portion of these damages have risen from columns.

This issue does not concern the IDR parameter, as it can illustrate a clear margin between CP and GI limit states. Nonetheless, some modifications can become very useful to overcome this insufficiency with local damage indices. For example, instead of using a weighted average, collapse can be defined as when the maximum damage in columns of a story reaches the value of 1.0. Also, it is possible to use a greater weighting coefficient for columns in Equation 12 which makes the collapse criterion more dependent on columns.

7.3 Statistical distribution of structural capacities at limit states

As the final part of results, in this section the statistical characteristics of distribution of structural capacities at IO, CP and GI levels will be presented. Based on the damage quantities calculated for IO and CP states, and also considering the criterion assumed for collapse of the structure, mean (σ), median (μ), and coefficient of variation (cv) for $S_a(T_1,\xi)/g$ at the three limit states are presented in the following tables:

Damage Measure	Base Case $S_a(T_1,\xi=0.05)/g$			Uncertain Case $S_a(T_1,\xi)/g$		
	σ	μ	cv	σ	μ	cv
IDR	0.592	0.564	28.0%	0.558	0.535	31.8%
D_{DR}	0.592	0.554	21.2%	0.558	0.573	35.0%
D_{RM}	0.592	0.499	40.5%	0.558	0.567	35.3%
D_{PA}	0.592	0.578	19.3%	0.558	0.569	33.3%
D_{KZ}	0.592	0.582	20.3%	0.558	0.560	29.5%
δ_f	0.609	0.557	46.8%	0.563	0.502	79.0%

Table 4: Statistical distribution of structural capacities at IO performance level.

Damage Measure	Base Case $S_a(T_1,\xi=0.05)/g$			Uncertain Case $S_a(T_1,\xi)/g$		
	σ	μ	cv	σ	μ	cv
IDR	2.910	2.634	36.8%	2.741	2.805	0.321
D_{DR}	2.910	2.605	37.7%	2.741	2.827	0.338
D_{RM}	2.437	2.250	33.9%	2.162	2.350	0.310
D_{PA}	2.910	2.625	38.1%	2.741	2.773	0.308
D_{KZ}	2.900	2.625	37.4%	2.741	2.808	0.296
δ_f	2.910	2.631	33.6%	2.739	2.770	0.364

Table 5: Statistical distribution of structural capacities at CP performance level.

Damage Measure	Base Case $S_a(T_1,\xi=0.05)/g$			Uncertain Case $S_a(T_1,\xi)/g$		
	σ	μ	cv	σ	μ	cv
IDR	3.312	3.075	37.8%	3.622	3.689	0.338
D_{DR}	2.950	2.625	37.6%	3.205	3.100	0.335
D_{RM}	2.437	2.250	33.9%	2.162	2.350	0.310
D_{PA}	3.125	2.925	38.5%	3.327	3.500	0.305
D_{KZ}	2.900	2.625	37.4%	3.105	3.100	0.293
δ_f	3.312	3.075	37.8%	4.065	4.400	0.321

Table 6: Statistical distribution of structural capacities at GI limit state.

Considering the results of Tables 4 to 6, the following points can be highlighted:

- Epistemic uncertainties have increased the structural capacities at CP and GI limit states, though for IO state it is generally the opposite.
- The coefficient of variation, which is an indicator of dispersion in data, vary among damage measures.
- Collapse capacities calculated by IDR are always greater than those of local indices, and equal or less than those related to δ_f.

8 CONCLUSIONS

- Use of various Damage Measures (or Engineering Demand Parameters) lead to creation of performance curves which are distinct in terms of interception with the vertical axis (seismic intensities), maximum structural capacities and variations in the curve's slope from linear stage to structural failure.
- The criterion which describes structural collapse has an essential role in interpretation of IDA curves based on local damage indices. As for the procedure used herein, it seems that a weighted average of 1.0 at story level cannot be an appropriate criterion for description of collapse. This issue is mainly due to the fact that damages occurred to columns and beams of a story do not have identical contribution to stability of the whole structure.
- The Damage Measures studied here display different levels of sensitivity to modelling uncertainties. These Measures are analytical functions that use different parameters as their inputs, and as the result, the dispersion of structural capacities calculated by each of them can show a different variability against epistemic uncertainties.

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