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Abstract	<p>Recently proposed capacity-based damage indices and parametric models for capacity curves are applied to frame steel buildings located in soft soils of the Mexico City. To do that, the seismic performance of 2D models of low-, mid- and high-rise buildings is assessed. Deterministic and probabilistic nonlinear static and incremental dynamic analyses are implemented. Monte Carlo simulations and the Latin Hypercube sampling technique are used. Seismic actions are selected among accelerograms recorded in the study area. Spectral matching techniques are applied, so that the acceleration time histories have a predefined mean response spectrum and controlled error. The design spectrum of the Mexican seismic code for the zone is used as target spectrum. The well-known Park and Ang damage index allows calibrating the capacity-based damage index. Both damage indices take into account the contribution to damage of the stiffness degradation and of the energy dissipation. Damage states and fragility curves are also obtained and discussed in detail. The results reveal the versatility, robustness and reliability of the parametric model for capacity curves, which allow modelling the nonlinear part of the capacity curves by the cumulative integral of a cumulative lognormal function. However, these new capacity-based damage index and capacity models have been tested for and applied to 2D frame buildings only; they have not been applied to 3D building models yet. The Park and Ang and the capacity-based damage indices show that for the analysed buildings, the contribution to damage of the stiffness degradation is in the range 66–77% and that of energy loss is in the range 29–34%. The lowest contribution of energy dissipation (29%) is found for the low-rise, more rigid, building. The energy contribution would raise with the ductility of the building and with the duration of the strong ground motion. High-rise frame buildings in soft soils of Mexico City show the worst performance so that the use of adequate braced frames to control the displacements could be recommended.</p>	

Keywords (separated by '-') Non-linear structural analysis - Parametric model - Monte Carlo simulation - Steel buildings - Damage assessment

Footnote Information



ORIGINAL RESEARCH PAPER

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3 **Capacity, damage and fragility models for steel**
 4 **buildings: a probabilistic approach**

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9 **Abstract** Recently proposed capacity-based damage indices and parametric models for
 10 capacity curves are applied to frame steel buildings located in soft soils of the Mexico City.
 11 To do that, the seismic performance of 2D models of low-, mid- and high-rise buildings is
 12 **AQ1** assessed. Deterministic and probabilistic nonlinear static and incremental dynamic anal-
 13 yses are implemented. Monte Carlo simulations and the Latin Hypercube sampling tech-
 14 nique are used. Seismic actions are selected among accelerograms recorded in the study
 15 area. Spectral matching techniques are applied, so that the acceleration time histories have
 16 a predefined mean response spectrum and controlled error. The design spectrum of the
 17 **AQ2** Mexican seismic code for the zone is used as target spectrum. The well-known Park and
 18 Ang damage index allows calibrating the capacity-based damage index. Both damage
 19 indices take into account the contribution to damage of the stiffness degradation and of the
 20 energy dissipation. Damage states and fragility curves are also obtained and discussed in
 21 detail. The results reveal the versatility, robustness and reliability of the parametric model
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 23 cumulative integral of a cumulative lognormal function. However, these new capacity-
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 29 for the low-rise, more rigid, building. The energy contribution would raise with the ducti-
 30 lity of the building and with the duration of the strong ground motion. High-rise frame

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31 buildings in soft soils of Mexico City show the worst performance so that the use of
 32 adequate braced frames to control the displacements could be recommended.

33 **Keywords** Non-linear structural analysis · Parametric model · Monte
 34 Carlo simulation · Steel buildings · Damage assessment

36 1 Introduction

39 The main purpose of this paper is to check the new damage index and the new capacity and
 40 fragility models, proposed by Pujades et al. (2015), when they are applied to steel
 41 buildings. In fact, this damage index and these parametric and fragility models have been
 42 tested only in a single simple reinforced concrete building; thus, the results of this paper
 43 will endorse the robustness, reliability and utility of these recent developments. Also, an
 44 important goal is to carry out a full probabilistic assessment of the seismic performance of
 45 low-, mid- and high-rise frame steel buildings in Mexico City. The method used by Vargas
 46 et al. (2013) to assess the seismic performance of a Reinforced Concrete (RC) building has
 47 been adopted; due to the regularity in plan and elevation, buildings are modelled as 2D
 48 frame structures in these works; applications to 3D building models await further research.

49 Concerning the new capacity model, the parametric model assumes that capacity curves
 50 are composed of a linear and a non-linear part. The linear part is defined by the initial
 51 stiffness or, equivalently, by a straight line whose slope (m) is defined by the fundamental
 52 period of vibration of the building. The non-linear part represents the degradation of the
 53 building and can be parameterized by means of the cumulative integral of a cumulative
 54 lognormal function and, therefore, it can be defined by two parameters, μ and σ ; the
 55 ultimate capacity point (S_{du} , S_{au}) provides the two last parameters of the five fully defining
 56 the capacity curve. Figure 1 shows an example of a capacity curve defined by these five
 57 parameters. The first derivative of the non-linear part of the capacity curve is also shown in
 58 this figure. This first derivative displays the cumulative lognormal function.

59 Concerning the new damage index and fragility model, on the basis of damage
 60 observations, many damage indices have been published that can be used to assess
 61 expected damage in buildings affected by earthquakes. These damage indices are related to
 62 degradation of the overall capacity of the structure to withstand the foreseen seismic loads,
 63 and they are usually defined on the basis of variation of specific parameters representing
 64 the strength and/or weakness of the building. Thus, for instance, damage indices based on
 65 displacement ductility were used by Powell and Allahabadi (1988) and by Cosenza et al.
 66 (1993). Bracci et al. (1989) and Bojorquez et al. (2010) focused on energy dissipation;
 67 Krawinkler and Zohrei (1983) paid attention to cyclic fatigue. Changes (increases) in the

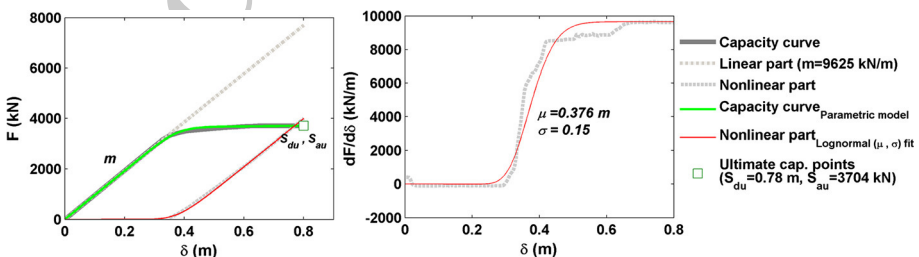


Fig. 1 Capacity curve as defined by five independent parameters



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68 natural period of the structure have also been used as damage indicators (DiPasquale and
69 Cakmak 1990); and Kamaris et al. (2013) focused on strength and stiffness degradation.
70 Other authors, such as Banon and Veneziano (1982), Park and Ang (1985), Roufaiel and
71 Meyer (1987) and Bozorgnia and Bertero (2001), connected the expected damage to
72 combinations of the above parameters. All these indices should be considered damage
73 pointers and properly fulfil the purpose for which they were developed. However, in many
74 cases, their calculation in practical applications involves Non Linear Dynamic Analysis
75 (NLDA), which has high computational costs. More recently, a new capacity-based
76 damage index was proposed by Pujades et al. (2015). This new damage index, which is
77 based on secant stiffness degradation and energy dissipation, was successfully calibrated
78 using a 2D model of a reinforced concrete frame buildings in such a way that it is
79 equivalent to the well-known Park and Ang damage index (Park et al. 1985; Park and Ang
80 1985) obtained by means of NLDA. The main advantage of the new index is that, once
81 calibrated, it can be obtained in an easy and straightforward way, directly from capacity
82 curves.

83 Concerning the probabilistic assessment of the seismic performance of frame steel
84 buildings in Mexico City, it is well known that variables involved in the seismic assess-
85 ment of structures have high uncertainties. These uncertainties can be organized into
86 aleatory (or random) and epistemic (or knowledge) uncertainties (Wen et al. 2003;
87 McGuire 2004; Barbat et al. 2011). Epistemic uncertainties are due to lack of knowledge
88 about models and/or parameters; aleatory uncertainty is inherent to random phenomena.
89 Uncertainties in the seismic actions and in the properties of the buildings are considered. In
90 relation to seismic actions, aleatory uncertainties are associated with the expected ground
91 motions, and, therefore, they cannot be controlled, but they can be estimated and addressed
92 through probabilistic approaches. In this research, uncertainties in seismic actions are
93 defined by means of a suite of accelerograms whose acceleration response spectra have
94 predefined mean and standard deviation; the design spectra for soft soils in the city of
95 Mexico (NTC-DF 2004) define the mean response spectrum. Regarding structures, aleatory
96 uncertainties are due to unawareness of the precise mechanical and geometrical properties.
97 Certainly, uncertainties in mechanical properties can be reduced by means of lab tests; in
98 this research, the uncertainty model used by Kazantzi et al. (2014) has been adopted; thus,
99 the mass, damping and other geometrical parameters are assumed to be deterministic, and
100 the strength and ductility of structural elements are considered in a probabilistic way.

101 Another important issue is how uncertainties propagate. Because of non-linearity,
102 uncertainties in the response strongly depend on the non-linear relations between inputs
103 and outputs. Thus, to take into account the effect of uncertainties in the response, in
104 deterministic approaches, seismic design standards recommend the use of reduced values
105 for strength of materials and increased actions, by means of safety factors. However, in
106 non-linear systems, it is well-known that the confidence levels associated with the response
107 may be different from those associated with the input variables (Vargas et al. 2013). Thus,
108 in the last two decades, the importance of performing probabilistic non-linear static
109 analysis (NLSA) (ATC-40 1996; Freeman 1998) and non-linear dynamic analysis (NLDA)
110 has been emphasized, (McGuire 2004) and, currently, there is a consensus that proba-
111 bilistic approaches are more suitable than deterministic ones, as they allow the incorpora-
112 tion of uncertainties, including confidence intervals, and thus provide more reliable
113 results. However, NLDA is assumed to be the most appropriate method for assessing
114 expected damage in structures subjected to dynamic actions (Vamvatsikos and Cornell
115 2002). Thus, when the capacity spectrum method (CSM) is used, it should be verified that
116 the results are consistent with those obtained from Nonlinear Incremental Dynamic



117 Analysis (NLIDA) (Mwafy and Elnashai 2001; Kim and Kurama 2008). In recent studies,
118 probabilistic static and dynamic approaches have been implemented using the Monte Carlo
119 simulation method (Fragiadakis and Vamvatsikos 2008, 2010; Vargas et al. 2013; Kazantzi
120 et al. 2014; Barbat et al. 2016). But, probabilistic analyses require a significant number of
121 NLIDAs and/or NLSAs, entailing a high computational cost. Therefore, it would be useful
122 to take advantage of simplified methods to compare the results obtained by means of
123 NLSA and NLIDA. An example of such a simplified approach is that proposed by Pujades
124 et al. (2015).

125 In this research, both static and dynamic analyses are performed by means of a prob-
126 abilistic approach that uses the Monte Carlo simulation method and the Latin Hypercube
127 Sampling (LHS) technique to optimize the number of samples. This fully probabilistic
128 approach can quantify the expected uncertainties in the response and in the expected
129 damage, produced by uncertainties in the material properties and the seismic actions. The
130 results show how uncertainties in the response and in the expected damage increase with
131 the severity of seismic actions. Moreover, it is shown how, static and dynamic approaches
132 provide consistent results. However, for the buildings analysed in this work, the consis-
133 tency is lower for high-rise buildings. This fact is attributed to the likely influence of higher
134 modes, which are not considered in the static analyses, as adopted herein. Finally, it is also
135 shown that the capacity parametric model and capacity based damage index also hold for
136 steel structures, so capacity curves can be represented by means of a simple model. The
137 expected damage and fragility curves can be analysed directly from capacity curves, in a
138 simple and straightforward way, thus avoiding the large amounts of computation involved
139 in dynamic simulations.

140 2 Buildings

141 2.1 Structural models

142 Three steel buildings are analysed in this paper; namely high- (13 stories), mid- (7 stories)
143 and low-rise (3 stories) buildings, with Special Moment Frames (SMF). Steel W type
144 sections (wide flange American section) are used for beams and columns, which are joined
145 by means of prequalified connections (ANSI/AISC 358-10 2010) of Fully Restrained (FR)
146 type. Buildings were designed as offices, on the basis of the provisions for the México City
147 area of NTC-DF (2004) and AISC-341-10 (2010) seismic codes. Buildings have rectan-
148 gular floors, 3 beams of 5 m, in the transverse direction, and 4 beams of 6 m in the
149 longitudinal direction. For each building, our focus will be on the central frame in the
150 longitudinal direction. The design of the SMFs satisfies the AISC criterion of strong
151 column-weak beam. Figure 2 shows a sketch of the three 2D-models (SMF 3, SMF 7 and
152 SMF 13).

153 NLSAs and NLIDAs were performed with Ruaumoko 2D software (Carr 2002). The
154 weight of the structure, as well as that of the architectural finishes and facilities, were
155 considered dead loads (DL), while live loads (LL) were established according to NTC-DF
156 (2004) provisions for office use. Total gravity loads for non-linear analysis are established
157 as 1.0 DL + 0.2 LL (PEER/ATC 72-1 2010). Beams and columns were modelled as
158 FRAME type members, with plastic hinges at their ends. Plastic hinges follow the Bi-
159 Linear Hysteresis rule, with hardening and strength reduction based on the ductility factor
160 [see Appendix A—Ruaumoko 2D (Carr 2002)]. Due to the limitations of the adopted

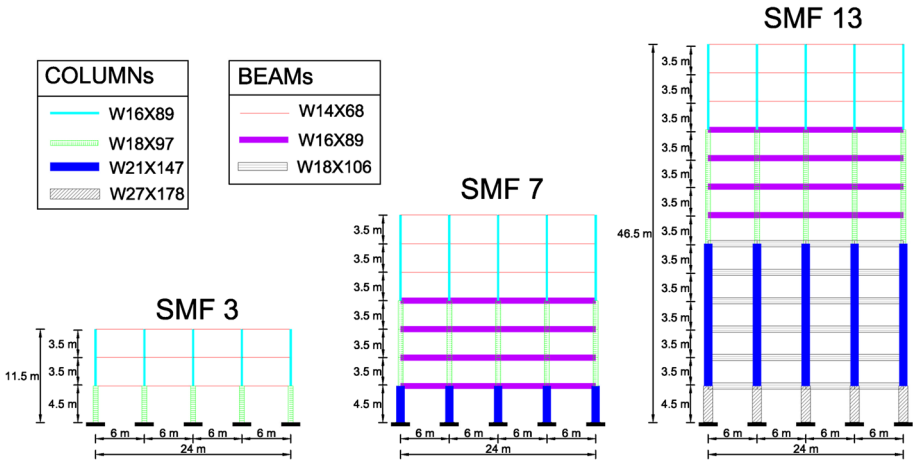


Fig. 2 2D building models

161 model, which only reproduces failure by bending moment and shear force, the interaction
 162 between moment and axial force is not considered. In addition, most of the damage for this
 163 type of buildings is expected to occur at the ends of the elements, mainly because of the
 164 combined effects of moment and shear. Therefore, the interaction of yield surface is
 165 defined for columns and beams by the diagram relating the bending moment with the
 166 rotation. Moreover, the values of strength and ductility for the hysteresis rule were calcu-
 167 lated according to the modified Ibarra–Medina–Krawinkler (IMK) model (Ibarra et al.
 168 2005; Lignos and Krawinkler 2011, 2012, 2013). This model establishes strength bounds
 169 on the basis of a monotonic backbone curve (Fig. 3a). The backbone curve is defined by
 170 three strength parameters (M_y = effective yield moment, M_c = capping moment
 171 strength—or post-yield strength ratio M_c/M_y —and $M_r = \kappa \cdot M_y$, $\kappa = 0.4$, residual
 172 moment) and by four deformation parameters (θ_y = yield rotation, θ_p = pre-capping
 173 plastic rotation for monotonic loading—difference between yield rotation and rotation at
 174 maximum moment, θ_{pc} = post-capping plastic rotation—difference between rotation at
 175 maximum moment and rotation at complete loss of strength—and θ_u = ultimate rotation
 176 capacity) (Lignos and Krawinkler 2011). The columns of the moment-resisting bays were
 177 assumed to be fixed at their bases. P–Delta effects were also considered. The panel zones
 178 were modelled by the rotational stiffness in the connections, obtained according to the

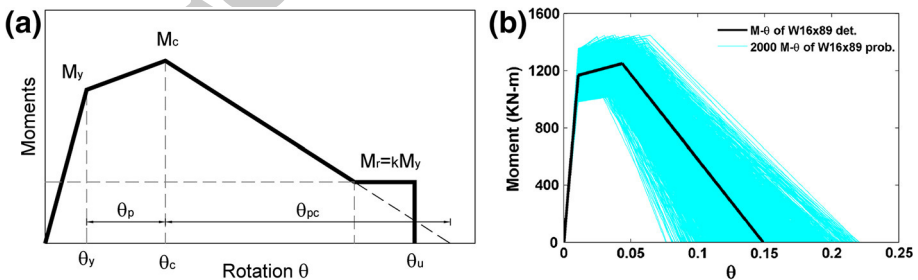


Fig. 3 **a** Modified IMK model: monotonic curve; **b** an example of the modified IMK model used in the structural section ($W16 \times 89$) of the SMF3 probabilistic models



179 model proposed by Krawinkler (1978) and presented in FEMA 355C (2000). In all cases,
 180 as recommended for steel structures, for the first and last vibration mode under consid-
 181 eration (SAC 1996), 2% Rayleigh damping was assumed. The fundamental periods of the
 182 models are 0.632, 1.22 and 1.92 s for SMF3, SMF7 and SMF13 buildings respectively.

183 2.2 Probabilistic variables

184 There are many sources of uncertainties in structural analysis. Even geometric properties,
 185 such as thickness, length and width of the structural elements or of the structure itself, can
 186 be considered probabilistic variables. Concerning mechanical properties, several paramet-
 187 ers can be considered in a probabilistic way, such as Young's modulus, ultimate strength,
 188 plastic modulus and so on. However, to make the probabilistic approach clearer and easier,
 189 in this study only a few properties are considered in a probabilistic manner. Thus, the
 190 probabilistic model for mechanical properties used by Kazantzi et al. (2014) has been
 191 adopted so that only uncertainties in strength and ductility are considered. In order to see
 192 the influence of uncertainties in mechanical properties on uncertainties in the response, an
 193 uncertainty analysis will also be performed. This analysis will show how the most
 194 important source of uncertainty is that due to seismic actions, although that due to
 195 mechanical properties may also be significant. Thus, in this study, the mass, damping and
 196 other geometrical parameters are assumed deterministic, and the strength and ductility of
 197 structural elements are considered in a probabilistic way.

198 Concerning strength, all the parameters of the modified IMK model can be obtained
 199 from three properties of the sections. That is, plastic modulus, Z , expected yield strength,
 200 f_y , and modulus of elasticity, E . Moreover, due to the fact that E and Z , for W sections,
 201 have low coefficients of variation (COV), and taking into account that E is directly related
 202 to f_y by means of the strain ϵ , whose value for steel is accurately determined, it is
 203 considered that uncertainty in f_y can take up the low uncertainties of E and Z , thus avoiding
 204 overestimations of uncertainties in the strength parameters. Notably, COV takes values
 205 between 1 and 3% (Bartlett et al. 2003) for E , and between 1 and 2% (Jaquess and Frank
 206 1999; Schmidt and Bartlett 2002) for Z ; uncertainties in f_y are higher. Thus, only f_y , is
 207 defined herein as a random variable for the strength. The mean (μ) value, standard deviation
 208 (σ) or COV and the assumed probability distributions for f_y are shown in Table 1.

209 The ductility of the structural sections are defined by the deformation parameters θ_y , θ_p
 210 and θ_{pc} of the modified IMK model; for W sections, these parameters can be determined by
 211 means of the following multi-variable empirical equations that were developed by Lignos
 212 and Krawinkler (2011, 2012, 2013):

$$\theta_y = (M_y/k_o)/L = (1.17 \cdot Z \cdot f_y/6 \cdot E \cdot I)/L \quad (1)$$

$$\theta_p = 0.0865 \cdot \left(\frac{h}{t_w}\right)^{-0.365} \cdot \left(\frac{b_f}{2 \cdot t_f}\right)^{-0.140} \cdot \left(\frac{L}{d}\right)^{0.340} \cdot \left(\frac{c_{unit}^1 \cdot d}{533}\right)^{-0.721} \cdot \left(\frac{c_{unit}^2 \cdot f_y}{355}\right)^{-0.721} \quad (2)$$

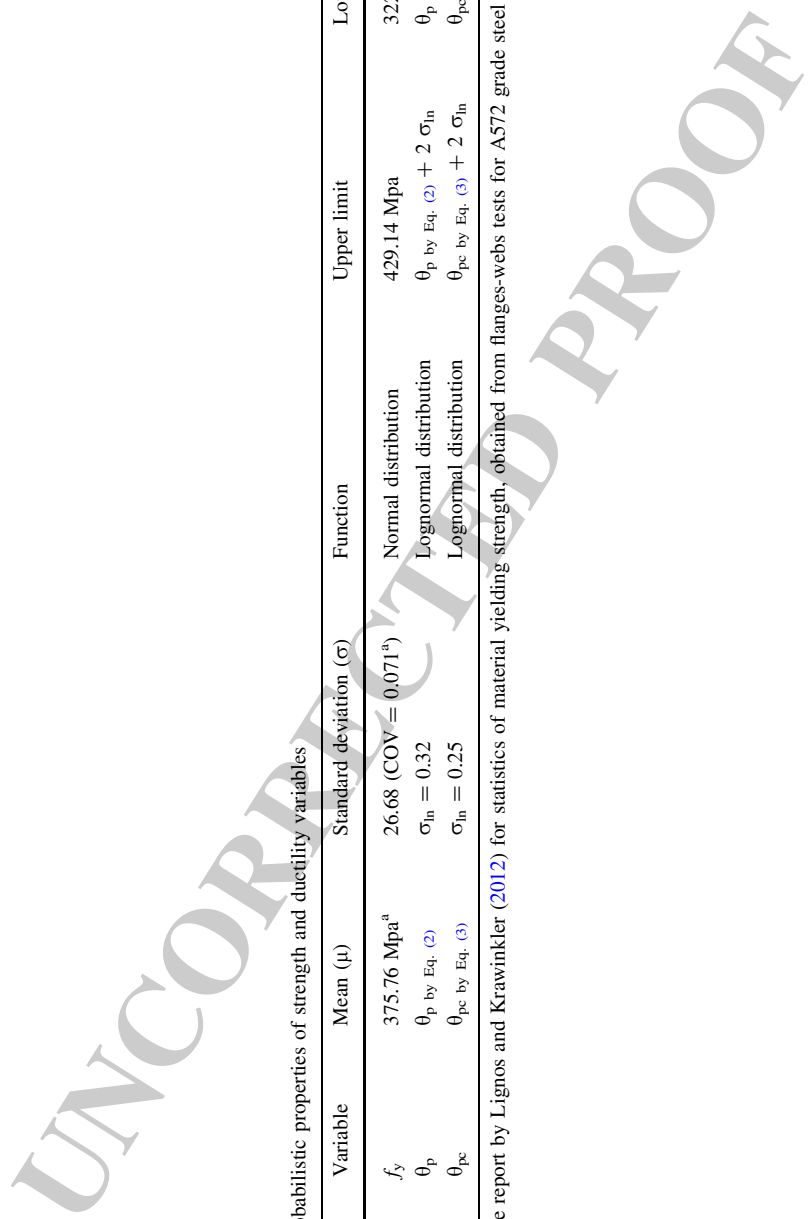
$$\theta_{pc} = 5.63 \cdot \left(\frac{h}{t_w}\right)^{-0.565} \cdot \left(\frac{b_f}{2 \cdot t_f}\right)^{-0.800} \cdot \left(\frac{c_{unit}^1 \cdot d}{533}\right)^{-0.280} \cdot \left(\frac{c_{unit}^2 \cdot f_y}{355}\right)^{-0.430} \quad (3)$$

$\sigma_{In} = 0.32$
 $\sigma_{In} = 0.25$

Table 1 Probabilistic properties of strength and ductility variables

Type	Variable	Mean (μ)	Standard deviation (σ)	Function	Upper limit	Lower limit
Strength	f_y	375.76 Mpa ^a	26.68 (COV = 0.071 ^{ab})	Normal distribution	429.14 Mpa	322.4 Mpa
Ductility	θ_p	θ_p by Eq. (2)	$\sigma_{\ln} = 0.32$	Lognormal distribution	θ_p by Eq. (2) + 2 σ_{\ln}	θ_p by Eq. (2) - 2 σ_{\ln}
Ductility	θ_{pc}	θ_{pc} by Eq. (3)	$\sigma_{\ln} = 0.25$	Lognormal distribution	θ_{pc} by Eq. (3) + 2 σ_{\ln}	θ_{pc} by Eq. (3) - 2 σ_{\ln}

^aBased on the report by Lignos and Krawinkler (2012) for statistics of material yielding strength, obtained from flanges-webs tests for A572 grade steel





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219 In these equations, k_0 is the initial elastic stiffness; I is the inertia moment; c_{unit}^1 and c_{unit}^2
 220 are coefficients for unit conversion. h/t_w is the ratio between the web depth and the
 221 thickness; L/d is the ratio between the span and the depth of the beam; $b_f/(2 \cdot t_f)$ is the
 222 width/thickness ratio of the beam flange, and σ_{in} is the standard deviation, assuming a
 223 lognormal fit of experimental data. Finally, the ultimate rotation capacity is estimated as
 224 $\theta_u = 1.5 \cdot (\theta_y + \theta_p)$, based on the recommendations of PEER/ATC 72-1 (2010). In this
 225 study, θ_y is considered a dependent variable of f_y , and θ_p and θ_{pc} are considered random
 226 variables with lognormal distributions. Mean (μ) values, standard deviations (σ_{in}) and
 227 function types used for θ_p and θ_{pc} are shown in Table 1. Uncertainties of θ_p (Eq. 2) and θ_{pc}
 228 (Eq. 3) also take into account the randomness of the dimensions of the W sections (Lignos
 229 and Krawinkler 2011, 2012, 2013), including uncertainties on I , h , d , t_w , b_f , t_f , and so on, as
 230 well as uncertainties on f_y .

231 Moreover, in order to avoid unrealistic samples in LHS simulations, both normal dis-
 232 tributions of f_y and lognormal distributions of θ_p and θ_{pc} were truncated at both ends, the
 233 lower and upper limits being determined by the mean value ± 2 times the standard
 234 deviation ($\mu \pm 2\sigma$). The purpose of this truncation is to avoid underestimates or overes-
 235 timates of the capabilities of the elements with samples without physical meaning.

236 In summary, a simplified probabilistic approach is proposed for this research. The method
 237 uses the modified IMK model for beams and columns, and uncertainties are concentrated on
 238 the variables f_y , θ_p and θ_{pc} . Thus, it is assumed that these three variables have a major
 239 influence on the linear and non-linear structural response of buildings. Besides, the use of
 240 these variables is recommended in the new codes for probabilistic seismic performance
 241 assessment of steel buildings (PEER/ATC 72-1 2010; FEMA P-58-1 2012).

242 2.3 Correlation analysis

243 Another important issue concerning sampling is the correlation among variables. Two
 244 types of correlation have been considered in this research: intra- and inter-element. The
 245 intra-element correlation is given by the relation among the three parameters simulated for
 246 the same hinge; these correlations can be derived from Eqs. (2) and (3) (Lignos and
 247 Krawinkler 2012) and are defined in Table 2.

248 The inter-element correlation is attributed to the consistency in workmanship and the
 249 material's quality among different element sections. Idota et al. (2009) and Kazantzi et al.
 250 (2014) proposed a value of 0.65 for the yield strength of beams and columns from the same
 251 production batch. Based on these studies, an inter-element correlation of 0.65 has been
 252 used herein for the same section type, and a null correlation is assumed for different
 253 sections.

254 2.4 Sampling

255 To better represent the physical randomness of the problem for each structural element
 256 (column or beam), a random sample of the three parameters (f_y , θ_p and θ_{pc}) is generated.

Table 2 Intra-element correla-
 tion for random variables of
 beams and columns

	f_y	θ_p	θ_{pc}
f_y	1	0	0
θ_p	0	1	0.69
θ_{pc}	0	0.69	1



257 Then, the properties of strength and ductility on the hinges of each element are estimated.
258 It is assumed that hinges at both ends of elements are the same. Thus, for instance, the
259 3-storey model, with 27 elements (15 columns and 12 beams) has 81 random variables; the
260 7-storey building with 63 elements (35 columns and 28 beams) has 189 random variables;
261 and the 13-storey model with 117 elements (65 columns and 52 beams) has 351 random
262 variables. In order to assess the seismic behaviour of these three buildings, with a prob-
263 abilistic approach, 200 NLSAs and 200 NLIDAs are performed for each structural model,
264 resulting in 600 NLSAs and 600 NLIDAs. The same structural models are used for both
265 structural analyses: static and dynamic. Figure 3b shows an example of the modified IMK
266 model used in the structural section (W16x89) of the SMF3 probabilistic models.

267 3 Seismic actions

268 To perform probabilistic IDAs, a set of accelerograms representing the characteristics of
269 the study area are needed. The way these acceleration time histories are obtained, is
270 explained first, and the method is then applied to the Mexico City to obtain probabilistic
271 response spectra and compatible acceleration time histories.

272 3.1 Method

273 In a first step, a set of random response spectra are generated by means of LHS simulations.
274 The response spectra meet the following conditions: (1) the mean value is a target spec-
275 trum, (2) the standard deviation in each period has a predefined value, and (3) the spectral
276 ordinates are correlated in such a way that spectra are realistic. As an example, Fig. 4
277 shows a set of five simulated response spectra. The fundamental periods of the studied
278 buildings are also depicted in this figure. Then, a spectral matching technique (Hancock
279 et al. 2006), is used to match the response spectrum of a real accelerogram to each one of
280 the simulated spectra. This way, a set of accelerograms that meet the above conditions can
281 be obtained. Moreover, if the seed accelerogram is chosen properly, the spectrum-matched
282 accelerograms are representative of the seismic actions expected in the area.

283 3.2 Probabilistic response spectra

284 In this study, the design spectrum for area III_b of the NTC-DF (2004) in Mexico City has been
285 taken as the target spectrum. Moreover, the standard deviation has been set to 5% for periods
286 from 0 to 2 s, corresponding to the range in which the periods of the buildings are situated, and
287 10% for periods greater than 2 s, thus controlling uncertainties in seismic actions.

288 3.3 Probabilistic acceleration time histories

289 A preliminary set of time histories was selected using the method proposed by Vargas et al.
290 (2013). A large database of 2554 accelerograms (three components) recorded in the
291 Mexico City area was used. Thus, four accelerograms with a relatively high compatibility
292 with the target spectrum were selected. Then a spectral matching technique was used to
293 improve the fit between response spectra of seed accelerograms and the target spectrum.
294 Figure 5 shows the seed accelerograms that have been selected, the matched ones and the
295 corresponding response spectra. This large database of Mexican accelerograms was

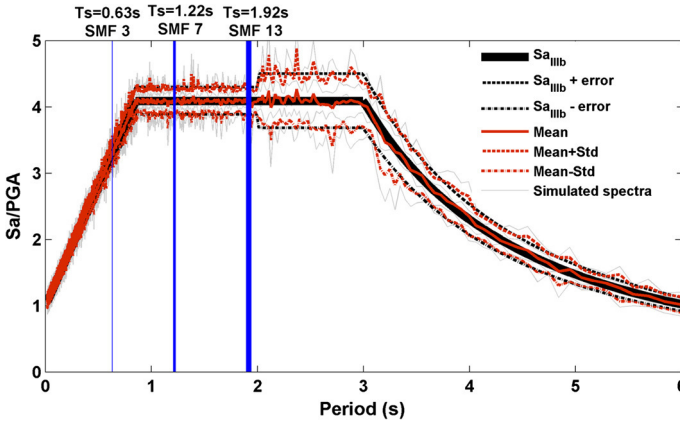


Fig. 4 Five simulated response spectra. Mean and standard deviation conditions are also shown. The five simulated spectra are used to match accelerogram acc1 (see Table 3)

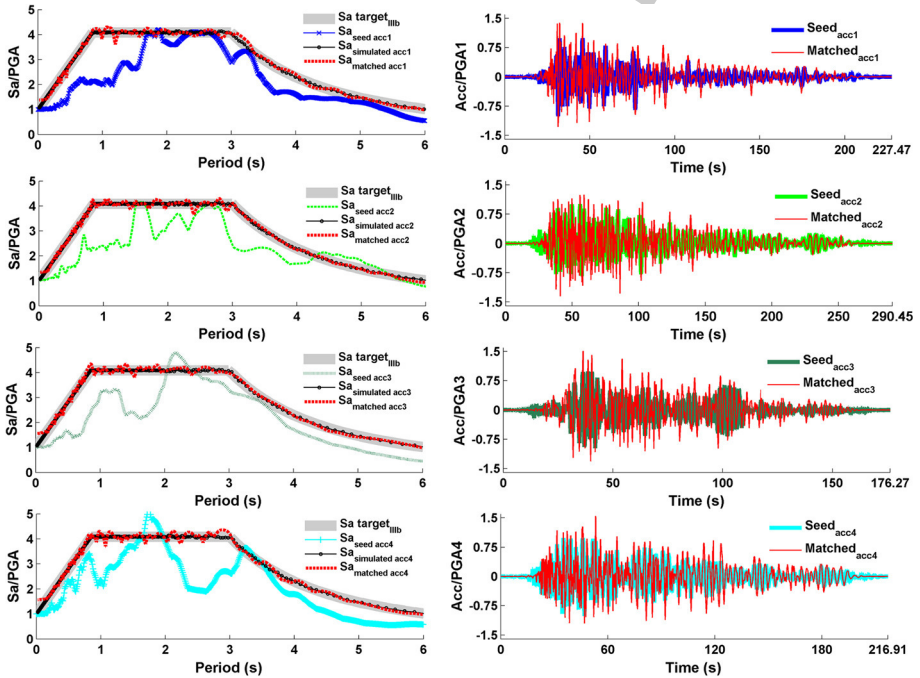


Fig. 5 Target spectrum and response spectra of the seed and matched accelerograms (right). Seed and matched accelerograms (left)

296 previously analysed by Diaz et al. (2015). Table 3 shows the characteristics of the four
 297 selected accelerograms and corresponding earthquakes. The PGA values are low, with a
 298 maximum PGA value of 49.6 cm/s^2 . This is due to the large epicentral distances of the
 299 earthquakes affecting Mexico City.

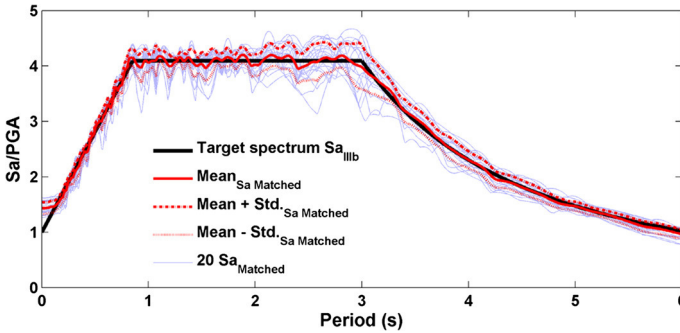


Fig. 6 Response spectra of the 20 accelerograms; mean and standard deviations are also depicted

300 No near-fault seismic actions are expected, as the seismic hazard of the city is dominated
 301 by the combined effects of distant, large earthquakes and soil amplification, leading to
 302 increased PGA values and long-duration acceleration records. These were the main causes of
 303 the destructive 1985 Michoacán earthquake. However, as shown below, the newly developed
 304 methods are valid for low and high PGA values, as both the capacity spectrum method and the
 305 NLIDA, allow any PGA value to be set for seismic actions affecting the buildings.

306 Spectral matching warrants the similarity between the shapes of the response spectra
 307 of the matched accelerograms and the code-provided design spectra, but both signals and
 308 spectra can be scaled to any PGA value, thus representing any level of seismic intensity
 309 well. In fact, in this study, PGA values have been set in the range between 0.05 and 0.7 g.
 310 Thus, for each seed accelerogram, the spectral matching technique was used to obtain 5
 311 new accelerograms meeting the probabilistic requirements described above. As a result, a
 312 set of 20 accelerograms were obtained. This number of accelerograms was considered
 313 adequate, as the Mexican seismic code (NTC-DF 2004) suggests that at least four
 314 **AQ3** accelerograms should be used. Twenty acceleration time histories was also considered a
 315 suitable number to deal with uncertainties in seismic actions, as they represent the pre-
 316 assumed probabilistic distributions well (see Figs. 4, 8). The whole set of response spectra
 317 corresponding to the 20 compatible accelerograms is shown in Fig. 6.

318 4 Probabilistic IDA

319 In this section, the influence that the randomness of the mechanical properties and the
 320 uncertainty of the seismic actions have on the uncertainties of the structural response is
 321 analysed and discussed. The analysis is shown for the low-rise buildings; similar con-
 322 clusions also hold for mid- and high-rise buildings.

323 4.1 Adequacy of the sampling

324 4.1.1 Mechanical properties

325 As pointed out above, 200 realizations of random structural parameters are used. This
 326 number has been determined in the following way. A number of random samples are
 327 generated according the truncated predefined probability density function (pdf). After
 328 every 20 new samples, the mean value and the standard deviation of the overall samples

Table 3 Characteristics of seed accelerograms selected by the Vargas et al. (2013) method

Acc.	Station	Date	Duration (s) ^a	Epicentre		Magnitude (Mw)	Component	PGA (cm/s ²)	Epicentral distance (km)	Azimut Star-Epi
				Latitude (N)	Longitude (W)					
Acc1	TH35	20/03/12	227.47	16.25	98.52	16	S00E	49.6	340.58	171.34
Acc2	AE02	30/09/99	290.45	15.95	97.03	16	N90W	21.3	442.48	150.64
Acc3	PCSE	11/01/97	176.27	17.91	103.04	16	S65W	14.6	442.84	248.35
Acc4	DM12	14/09/95	216.91	16.31	98.88	22	N00E	19.3	347.79	176.20

^aDuration refers to the total length of the accelerogram, including added time before and after event recording

329 are obtained. For more than 200 samples no significant variations are obtained in their
 330 mean value and standard deviation so that 200 has been considered an adequate number of
 331 samples representing the predefined truncated pdf. In fact, the LHS technique avoids
 332 duplicating case combinations, so that fewer samples adequately represent the target pdf
 333 (Iman 1999). Moreover, other authors have also used 200 probabilistic models to assess the
 334 seismic performance of buildings (Fragiadakis and Vamvatsikos 2008; Kazantzi et al.
 335 2008). Figure 7 shows the target normal and normal truncated pdfs together with the
 336 histogram of the 200 samples for the f_y random variable. A good agreement between
 337 histogram and the target pdfs can be seen. Similar plots can be depicted for the other
 338 random variables.

339 4.1.2 Seismic actions

340 For each probabilistic IDA, only 20 accelerograms are used. In order to see that 20 time
 341 histories adequately represent the foreseen uncertainties, so that actually 20 samples are
 342 sufficient for the probabilistic approach, the following analysis is performed. In fact,
 343 uncertainties in each acceleration time history affects all the periods of the response
 344 spectrum, that is, the response is affected by the uncertainty at all the periods. For each
 345 period, these uncertainties have been predefined by means of a normal pdf function that has
 346 the target spectrum as a mean value and a predefined standard deviation, which is 5%, in
 347 the period range 0–2 s, and 10%, in the period range 2–6 s.

348 To illustrate how these distributions are well fulfilled by the 20 accelerograms, Fig. 8a, b
 349 have been obtained as follows. For each one of the twenty response spectra matched by the
 350 seed accelerograms, the simulated random values, at each period, have been normalized by
 351 the value of the mean spectrum, in such a way that the normalized samples have a unit mean
 352 and the predefined standard deviation. Figure 8a corresponds to the samples in the period
 353 range (0–2) s and Fig. 8b corresponds to the period range [2–6] s. It can be seen how the
 354 twenty selected accelerograms adequately represent the predefined uncertainties with a unit
 355 mean (value of the mean target spectrum), and 0.05 and 0.1 standard deviations, respectively
 356 for the short and long period ranges. This way it can be seen how the 20 accelerograms
 357 adequately represent the predefined mean values and foreseen uncertainties. Moreover,

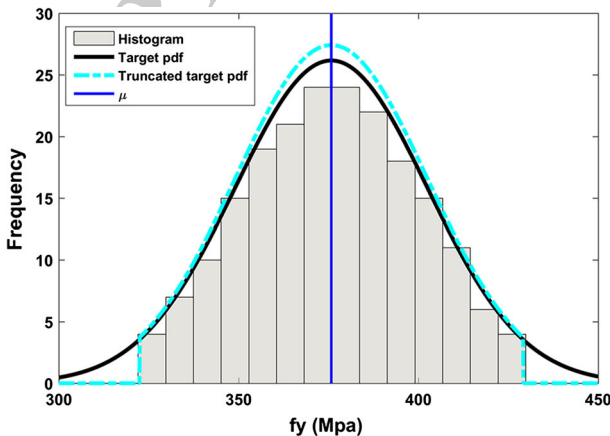


Fig. 7 Histogram of the 200 samples of the f_y and corresponding scaled pdf target functions

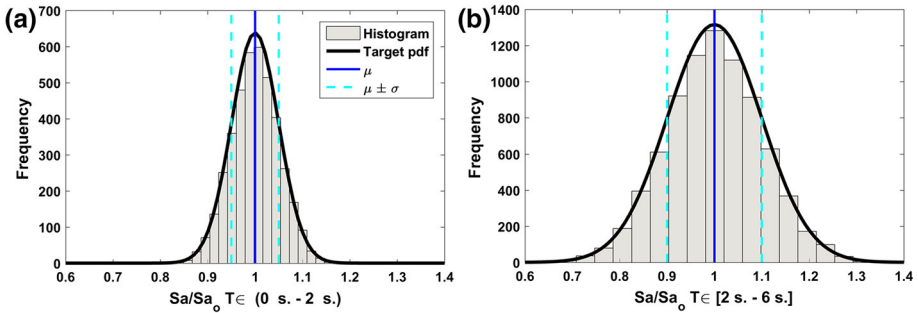


Fig. 8 Histogram of the samples used to define the seismic actions in a probabilistic way. Scaled target pdf functions are also shown. **a** In the period range (0–2) s. **b** In the period range (2–6) s (see also explanation in the text)

358 several probabilistic approaches in the literature (Kazantzi et al. 2008; Asgarian et al. 2010;
359 Celarec and Dolšek 2013; Vargas et al. 2013) use suites of 15–20 accelerograms. Besides,
360 Vamvatsikos (2014) proposes to limit the computational cost of probabilistic IDA evaluations
361 reducing the size of the ground motion records. Thus, if an incremental sampling
362 technique (Sallaberry et al. 2008; Vamvatsikos 2014) or some justified criterion is used, the
363 time histories can be reused. In this research it has been assumed that each accelerogram can
364 be reused, mainly because of the two following reasons: (1) as shown above, (see Fig. 8a, b)
365 the probabilistic spectral matching technique warrants the pre-assumed probability distribu-
366 tions in the uncertainties of the 20 seismic actions (see also Fig. 6), and (2) on the basis of
367 the principle that, all the records in the suite have the same probability of occurrence.

368 4.2 Uncertainty in the response

369 A total of 200 SMF3 structural models with the variables obtained from LHS Monte Carlo
370 simulations and the set of 20 compatible seismic actions are used. The analyses are
371 performed in such a way that the influence of the mechanical properties (f_y , θ_p , θ_{pc}) and the
372 impact of the seismic actions can be analysed separately.

373 NLIDA has been performed for different PGAs covering the range between 0.05 and
374 0.7 g, with PGA increments of 0.05 g. The following cases are analysed. First, the building
375 is considered as deterministic while the seismic action is considered as probabilistic by
376 using the 20 matched accelerograms; then the seismic action is considered as deterministic
377 by using the acc1 (see Table 3 and Fig. 5), matched to the selected target spectrum as
378 explained above. Thus, the following five cases are considered: (1) the building is con-
379 sidered deterministic and seismic actions probabilistic; (2) seismic action deterministic and
380 building probabilistic by considering uncertainties in the three mechanical properties (f_y ,
381 θ_p and θ_{pc}). In the following cases, the seismic action is considered in a deterministic way
382 and only uncertainties for one of the mechanical properties are considered according to the
383 following cases: (3) f_y , (4) θ_p and (5) θ_{pc} . In all these five cases and for each PGA, the
384 standard deviation (σ) in the structural response is computed; the roof displacement δ is
385 considered a control variable of the response. Figure 9 shows the results obtained. In
386 addition to the uncertainties in the roof displacement for the five cases described above, the
387 overall uncertainty is shown in this figure. This total uncertainty is obtained using the well-
388 known quadratic composition (Vargas et al. 2013). As expected, uncertainties due to
389 uncertainties in θ_p and θ_{pc} are small compared to those induced by uncertainties in f_y ; but

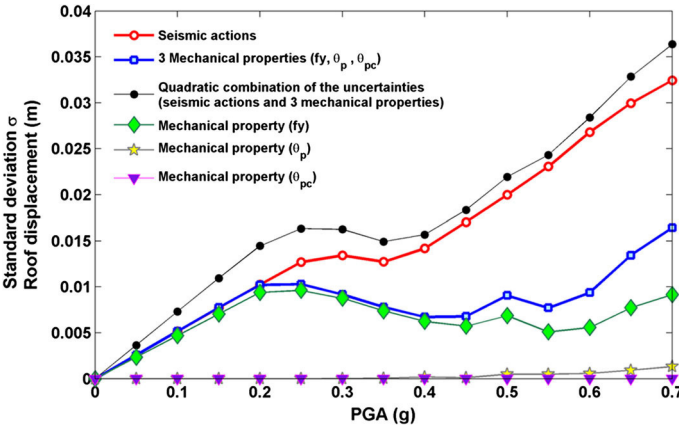


Fig. 9 Uncertainties in the roof displacement for the SMF3 building (see the discussion in the text)

390 uncertainties due to θ_p and θ_{pc} have a significant influence when they are combined with
 391 those due to f_y . The influence of uncertainties in seismic actions is clearly dominant.
 392 Probably, with the exception of θ_{pc} , uncertainties in the response tend to increase with
 393 increasing seismic actions. Similar results, concerning the influence of uncertainties in
 394 seismic actions and the increase of uncertainties with increasing seismic actions, were
 395 reported for reinforced concrete buildings in Vargas et al. (2013). The increase of
 396 uncertainties with increasing actions may be attributed to the fact that, for increasing PGA,
 397 the damage also increases, and the structural system becomes unstable, in the sense that
 398 small input variations produce considerable differences in the output.

399 5 Parametric model

400 In this section, the parametric model for capacity curves (Pujades et al. 2015) is applied.
 401 Deterministic and probabilistic cases are analysed. Mean values of strength-ductility of the
 402 sections are used for the deterministic approach and, as pointed out above, 600 models
 403 generated by LHS Monte Carlo techniques are used for the probabilistic approach.

404 5.1 Capacity curves

405 Capacity curves have been obtained by means of adaptive pushover analysis (PA) (Sat-
 406 yarno 2000) as implemented in the Ruaumoko software (Carr 2002). This method was
 407 shown to be independent from the initial loading pattern, as it adapts this pattern at each
 408 step of the PA, according to the deformation of the structure. The ultimate capacity is
 409 established when one of the following criteria is fulfilled. (1) ω^2 is less than $10^{-6} \omega^2$ at the
 410 first step, being ω the tangent fundamental natural frequency in the Modified Rayleigh
 411 Method; (2) the Newton–Raphson iteration is not achieved within a specified maximum
 412 number of cycles; (3) the stiffness matrix becomes singular and (4) a specified maximum
 413 structure displacement is reached. In the NLSAs of the studied buildings, a large number of
 414 cycles for the Newton–Raphson method has been considered. Moreover, a large maximum
 415 limit for the structure displacement has been considered. Thus, it is expected that failure

416 criteria be related to criteria (1) or (3) It is worth noting that the failure criterion is usually
 417 fulfilled when plasticization occurs in all the pillars of a story.

418 Figure 10 shows the obtained capacity curves. For comparison purposes the 5th, 50th
 419 and 95th percentiles are used. The following steps have been carried out to obtain a specific
 420 nth percentile: (1) capacity curves are interpolated/extrapolated in such a way that they are
 421 defined at the same points in the same interval; a fixed small displacement increment, $\Delta\delta$,
 422 is used to this end and the interval between 0 and the maximum ultimate displacement is
 423 used; (2) for each spectral displacement, ordinates are sorted from lowest to highest values
 424 (3) the nth percentile is computed at each spectral displacement (4) the ultimate dis-
 425 placement of the nth percentile is set to the nth percentile of the ultimate displacements.
 426 The 5th, 50th and 95th percentiles, computed this way, are shown in Fig. 10. Deterministic
 427 capacity curve and the 200 individual probabilistic capacity curves are also shown in this
 428 figure.

429 The 50th percentile curves (median) match the deterministic curves well, although the
 430 matching is better for SMF3 and SMF7 models. Differences between deterministic and
 431 median capacity curves are in the non-linear zone and they can be attributed mainly to non-
 432 linearity of the structural response. The fact that individual points of the median curve
 433 correspond to different capacity curves can also contribute to these differences.

434 5.2 Capacity model

435 The parametric model for capacity curves/spectra is well-described in Pujades et al. (2015).
 436 To test this model, capacity spectra have been preferred rather than capacity curves.

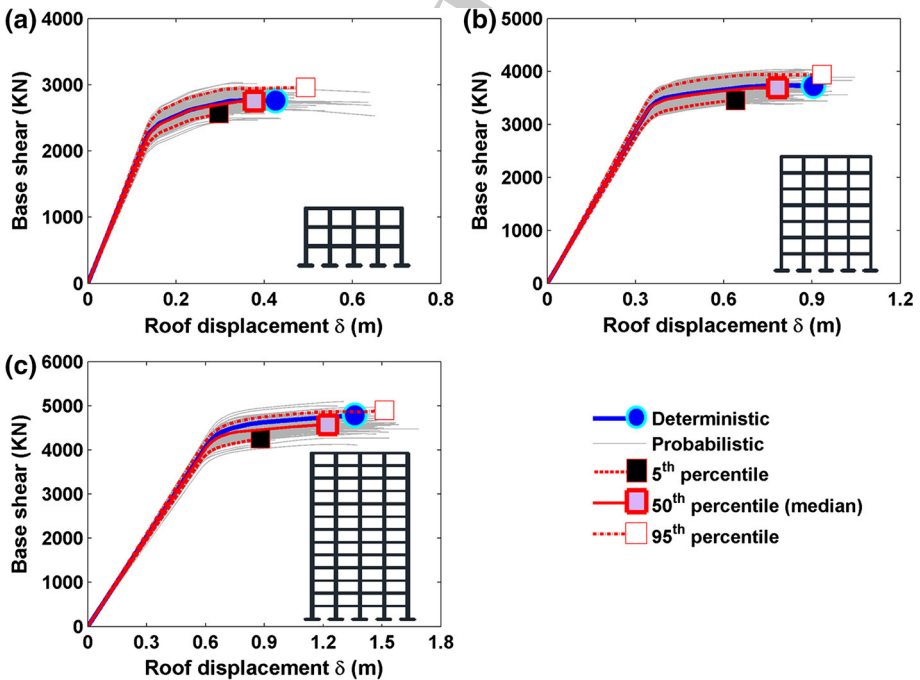


Fig. 10 Deterministic, probabilistic and percentiles of the capacity curves. a SMF 3, b SMF 7 and c SMF 13



437 Table 4 displays the weights, w_i and the normalized amplitudes $\Phi_i I$, at level i , for the first
438 natural mode. Table 5 shows the total weight, W , of the building and the period, T_1 , modal
439 participation factor, PF_1 , and modal mass coefficient, α_1 , for the first natural mode. Note
440 that $\Phi_{roof} I$, PF_1 and, α_1 , are used to transform capacity curves into capacity spectra (ATC
441 40 1996).

442 The five parameters that fully define the capacity spectrum are the initial slope (m), the
443 mean value (μ) and the standard deviation (σ) of the lognormal function and the ultimate
444 capacity point (S_{du} , S_{au}). m is related to the initial stiffness and to the period of the
445 fundamental mode of vibration; the cumulative lognormal function, defined by μ and σ , fits
446 the normalized first derivative of the non-linear part of the capacity spectrum.

447 Figure 11 displays the model as applied to the median capacity spectra of the three
448 buildings. Capacity spectra, together with their linear and non-linear parts, are shown
449 (upper part); first derivatives are shown in the lower part of this figure. Figure 12 shows the
450 individual and the deterministic capacity spectra; the obtained fits are also displayed.

451 The five parameters of the deterministic case and 5th, 50th and 95th percentiles are
452 given in Table 6. The mean values of the error vectors (% mean error) defined by the
453 difference, in percentage, between capacity spectra and the corresponding fit, are also
454 provided in this table. Mean errors are very small (always below the 3%). Note the likeness
455 between the parameters of the deterministic and 50th percentile capacity spectra.

456 6 Damage

457 An important issue related to seismic design of new buildings and, specially, related to
458 seismic risk assessment of existing structures and facilities is the expected damage. A
459 widely used damage index is the Park and Ang damage index (Park 1984; Park and Ang
460 1985; Park et al. 1985, 1987). We refer to this damage index as DI_{PA} . According to the
461 Park and Ang studies, structures are damaged because of the combined effects of dis-
462 placements in the nonlinear range due to their response to large stresses and of cyclic drifts
463 in response to cyclic strains. Therefore, damage assessment must take into account also
464 repeated cyclic loads/unloads, in addition to maximum structural response. Displacements
465 in the nonlinear range are related to stiffness degradation and cyclic loadings are related to
466 energy losses. This idea is based on the damage index proposed by Pujades et al. (2015),
467 which is also based on two functions related to stiffness degradation and to energy loss; but
468 now, these functions are computed, in a straightforward way, from capacity curves or
469 capacity spectra. We refer to this new capacity-based-damage index as DI_{CC} . DI_{PA}
470 is implemented in many computer programs for structural analysis and it is computed by
471 means of Non-Linear-Incremental-Dynamic-Analysis (NLIDA). The Ruaumoko 2D pro-
472 gram has been used to perform NLIDA and compute DI_{PA} . Further details on DI_{PA} can be
473 found in the Ruaumoko 2D technical manual (Carr 2002).

474 In this section, DI_{PA} and DI_{CC} are computed for the analysed steel buildings. Notice
475 that, according to Pujades et al. (2015) DI_{PA} is needed to calibrate the relative contribution
476 to damage of the stiffness degradation and of the energy loss.

477 6.1 Park and Ang damage index (DI_{PA})

478 Ruaumoko 2D is used to compute DI_{PA} through NLIDA. Notably, the failure or ultimate
479 point in the NLIDA is defined by the first roof displacement that exceeds the ultimate

Table 4 Weights, w_i , and normalized amplitude of the first natural mode, ϕ_i/l

Storey	1	2	3	4	5	6	7	8	9	10	11	12	13
SMF3													
w_i (KN)	885.9	881.4	605.6										
ϕ_i/l	0.4	0.775	1										
SMF7													
w_i (KN)	902.6	890.6	890.6	889.6	881.4	881.4	605.6						
ϕ_i/l	0.133	0.313	0.489	0.647	0.803	0.929	1						
SMF13													
w_i (KN)	924.5	909.7	909.7	909.7	909.7	903.2	890.6	890.6	890.6	889.6	881.4	881.4	605.6
ϕ_i/l	0.057	0.135	0.219	0.303	0.384	0.463	0.563	0.664	0.755	0.832	0.906	0.965	1

Table 5 Total weight, W , and period, T_1 , modal participation factor, PF_1 , and modal mass coefficient, α_1 , for the first natural mode

Building	W (kN)	T_1 (s)	PF_1	α_1
SMF3	2372.9	0.63	1.286	0.891
SMF7	5941.8	1.22	1.350	0.805
SMF13	11,396.3	1.92	1.397	0.754

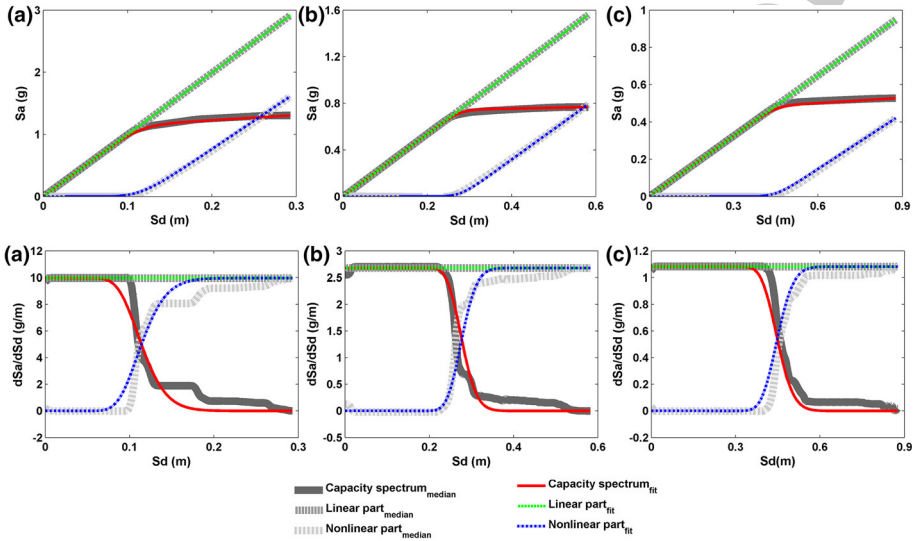


Fig. 11 Capacity spectrum, linear part and non-linear part (up) and corresponding first derivatives (down). Fits for the 50th percentile of the probabilistic capacity spectra are also shown. **a** SMF 3, **b** SMF 7 and **c** SMF 13

480 displacement of the corresponding capacity curve. Usually at this point, DI_{PA} is about 1,
 481 which confirms a failure condition.

482 To obtain probabilistic DI_{PA} with NLIDA, the suite of 20 accelerograms, whose
 483 response spectra have controlled mean and standard deviation, are used as follows.
 484 Accelerograms in the suite are organized and they are numbered between 1 and 20. Then,
 485 in each of the 200 IDA, an integer random number, uniformly distributed between 1 and
 486 20, is generated. The accelerogram having assigned this random number is used for the
 487 corresponding IDA analysis. The adequacy of this procedure for the purpose of this study
 488 has been also discussed above (see Sect. 4.1.2 and Fig. 8). To obtain DI_{PA} in a deter-
 489 ministic way, the mean of the four matched accelerograms, shown in Fig. 5, is used. This
 490 way a deterministic and 200 probabilistic functions, linking the roof displacement, δ , and
 491 the Park and Ang damage index, DI_{PA} are obtained. Again, the 5th, 50th and 95th per-
 492 centiles are used for discussion. The procedure to obtain these percentile curves has been
 493 briefly explained above. Figure 13 shows the results obtained for the SMF 3, SMF7 and
 494 SMF 13 building models.

495 For the deterministic NLIDAS, the mean of the four matched accelerograms shown in
 496 Fig. 5 is used. The δ - DI_{PA} functions for the studied buildings are shown in Fig. 13.
 497 Observe how deterministic DI_{PA} s are lower than the probabilistic 50th percentile. Because

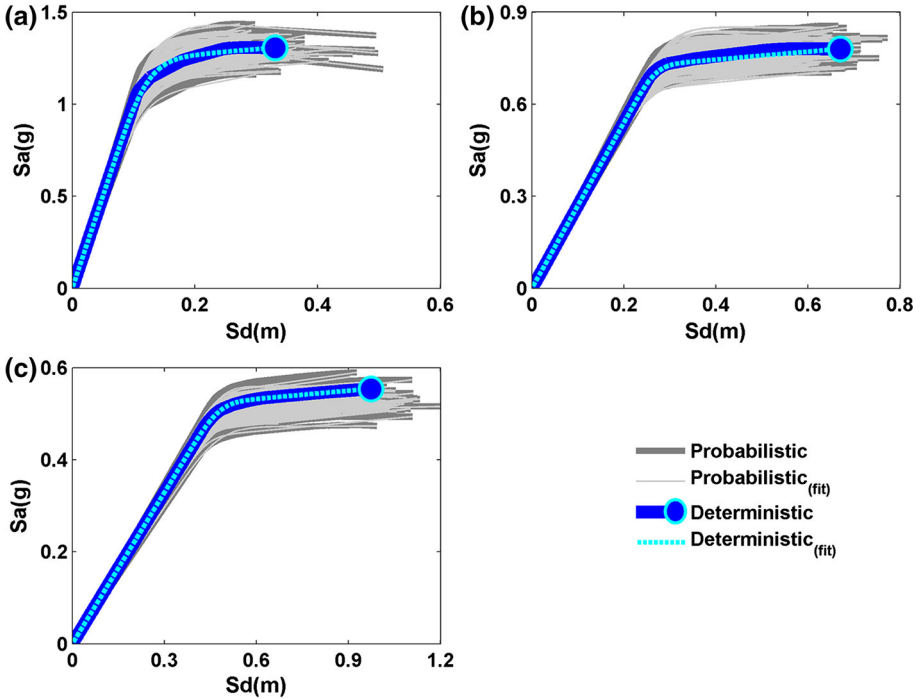


Fig. 12 Probabilistic capacity spectra and fits. The deterministic case is also shown. **a** SMF 3, **b** SMF 7 and **c** SMF 13

of nonlinearity of the structural response, the use of mean, median or characteristic values does not guarantee to get mean, median or characteristic responses. This fact highlights the importance of probabilistic approaches in front of the more frequently used deterministic approaches. Note that, in the case of Fig. 13, the use of mean values, both of the seismic actions and strength parameters, leads to un-conservative results, which emphasizes, even more, the importance of probabilistic approaches.

6.2 Capacity-based damage index (DI_{CC})

DI_{CC} is based on the combination of a stiffness degradation function, $K(\delta)$, and an energy dissipation function, $E(\delta)$. The computation of these functions is well-described in Pujades et al. (2015). However, for clarity, a basic explanation of how these functions are defined is also given herein. $E(\delta)$ is defined by the cumulative integral of the non-linear part of the capacity curve; the obtained function is then normalized, in abscissae and in ordinates, to obtain the normalized $E_N(\delta_N)$, ranging between 0 and 1 and also taking values between 0 and 1. $K(\delta)$ is defined by the ratio between the ordinates and abscissae of the non-linear part of the capacity curve; again, normalizing in abscissae and in ordinates, the $K_N(\delta_N)$ function is obtained. DI_{CC} is defined by the following equation:

$$DI_{CC}(\delta_N) = \alpha K_N(\delta_N) + (1 - \alpha)E_N(\delta_N) \cong DI_{PA} \quad (4)$$

where $E_N(\delta_N)$ and $K_N(\delta_N)$ are the normalized energy and stiffness functions defined above, and α is a parameter that defines the relative contributions to the damage index of the

Table 6 Parameters of the capacity model for the deterministic capacity curve and for the 5th, 50th and 95th percentiles of the probabilistic capacity spectra

	SMF3					SMF7					SMF13							
	m (g/m)	Sdu (m)	Sau (g)	μ	σ	%Mean error	m (g/m)	Sdu (m)	Sau (g)	μ	σ	%Mean error	m (g/m)	Sdu (m)	Sau (g)	μ	σ	%Mean error
Deterministic	10.05	0.33	1.31	0.37	0.25	0.17	2.70	0.67	0.78	0.40	0.10	0.77	1.10	0.97	0.55	0.49	0.10	0.06
5th percentile	9.44	0.23	1.21	0.48	0.10	0.25	2.55	0.48	0.72	0.58	0.15	0.65	1.04	0.63	0.49	0.70	0.05	0.01
Median	9.96	0.29	1.30	0.39	0.20	1.18	2.68	0.58	0.77	0.48	0.10	0.55	1.08	0.88	0.53	0.51	0.10	0.93
95th percentile	10.67	0.38	1.40	0.30	0.15	2.74	2.84	0.69	0.83	0.39	0.10	0.84	1.12	1.09	0.57	0.46	0.15	1.26

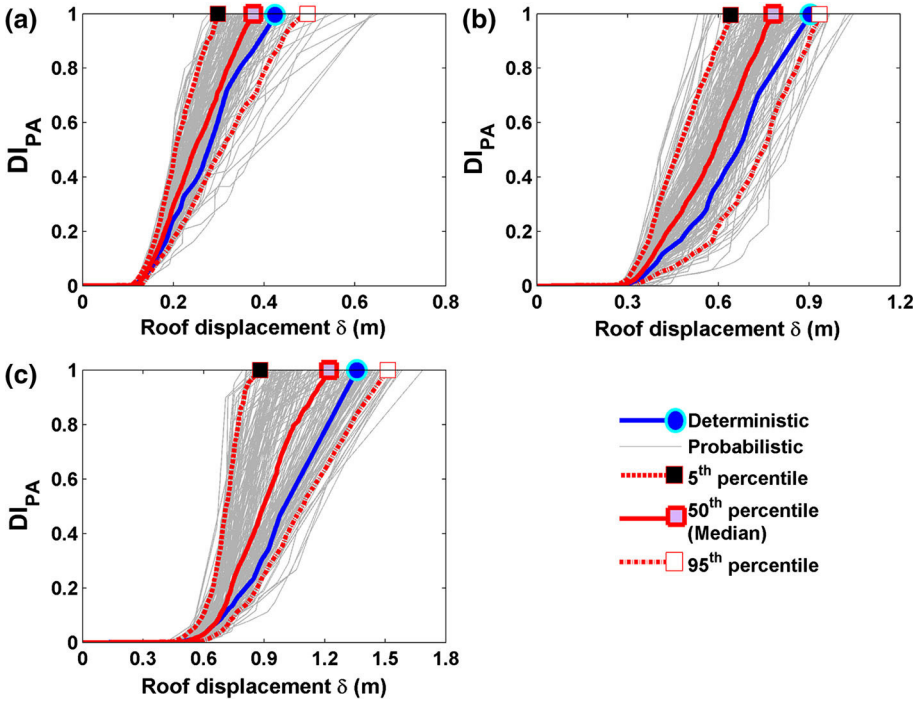


Fig. 13 δ - DI_{PA} functions obtained with NLIDAs for: a SMF 3, b SMF 7 and c SMF 13

517 stiffness degradation and that of the energy loss. The specific value of this parameter α , for
 518 a given seismic action, is calibrated by means of a least squares procedure applied to
 519 Eq. (4). This way this new damage index, DI_{CC} , is equivalent to DI_{PA} . Specific examples
 520 of $E_N(\delta_N)$, $K_N(\delta_N)$, $DI_{CC}(\delta_N)$ and $DI_{PA}(\delta_N)$ are shown below, in the following section
 521 (Fig. 14), where the results of the calibration of Eq. (4) are discussed.

522 6.3 Results and discussion

523 For the three analysed buildings, the calibration is illustrated for the median capacity
 524 curves using median DI_{PA} s. Thus, the $K_N(\delta_N)$, $E_N(\delta_N)$ and $DI_{PA}(\delta_N)$ functions are used to
 525 calibrate the parameter α , by means of a least squares fit of Eq. (4). α values are 0.71, 0.66
 526 and 0.67 for SMF3, SMF7 and SMF13 buildings respectively. Figure 14 shows these three
 527 cases. Undoing the normalization procedure these functions can be represented as func-
 528 tions of the roof displacements δ . Figure 15 shows $DI_{PA}(\delta)$ and $DI_{CC}(\delta)$ for the deter-
 529 ministic case and for the 5th, 50th and 95th percentiles capacity curves. The obtained
 530 values of α are in the range between 0.66 and 0.71, which is also similar to the range
 531 reported by Pujades et al. (2015) for reinforced concrete buildings. Thus, DI_{PA} (median) is
 532 well-represented by the new damage index DI_{CC} (median) obtained directly from the
 533 capacity curves. As explained above, the value of α is directly related to the relative
 534 contribution to damage of the secant stiffness degradation, while $(1 - \alpha)$ corresponds to
 535 the relative contribution of the energy loss. In the case of the median DI_{CC} of Fig. 15,
 536 contributions to damage of the stiffness degradation are in the range 66–71%, while the
 537 contribution of the energy loss is in the range 29–34%.

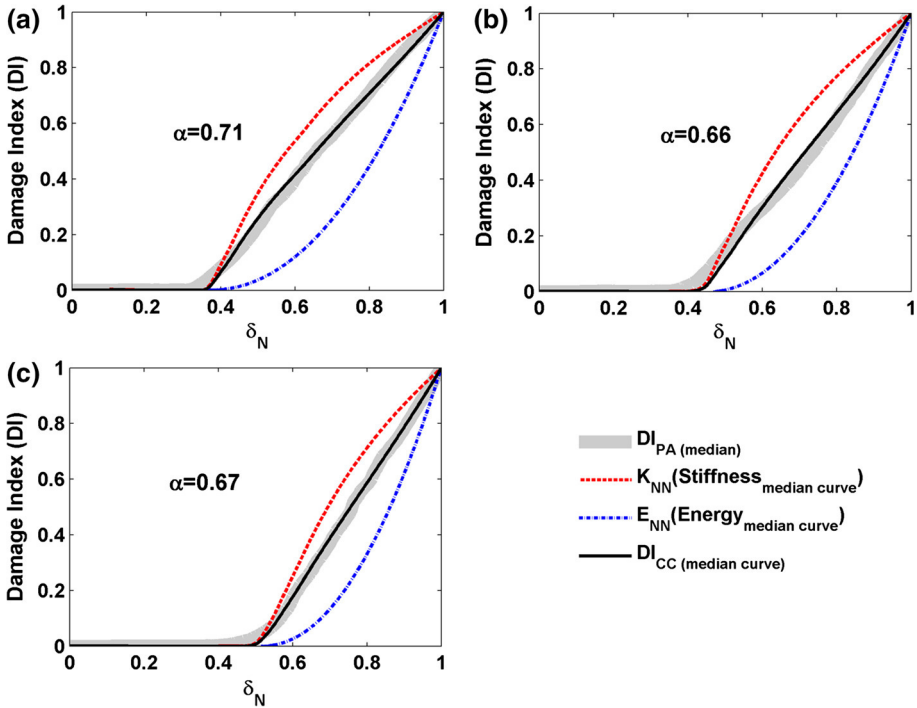


Fig. 14 Energy and stiffness degradation functions and calibration of the $DI_{CC}(\delta_N)$ for the median capacity curve. **a** SMF 3, **b** SMF 7 and **c** SMF 13

538 7 Fragility

539 Fragility curves and damage probability matrices are widely used in earthquake engi-
 540 neering (FEMA 2016; Milutinovic and Trendafiloski 2003; Lagomarsino and Giovinazzi
 541 2006). Porter (2017) is a nice tutorial for beginners (see also Porter et al. 2007). Details of
 542 the construction of fragility curves, in the framework of our research, are explained well in
 543 Lantada et al. (2009, 2010), Vargas et al. (2013) and Pujades et al. (2012, 2015). In this
 544 section, the basics of fragility curves, damage probability matrices and mean damage state
 545 are described first; then, the specific damage states thresholds used are introduced; finally,
 546 the obtained results are given and discussed.

547 7.1 Basics

548 In the earthquake engineering context, for a given damage condition or damage state, i , and
 549 for a level of seismic intensity measure, IM , the fragility curve, $F_i(IM)$, is defined as the
 550 probability that this damage state be exceeded, given the seismic intensity SI . Thus,
 551 fragility curves are usually given as functions of a variable (SI) linked to the severity of
 552 the seismic action such as, for instance, spectral displacement, PGA or macroseismic intensity,
 553 among others. The spectral displacement, S_d , is used herein. Fragility curves are com-
 554 monly modelled by means of cumulative lognormal functions defined by two parameters,
 555 μ_i and β_i . μ_i is the median of the lognormal function and is known as i -damage state
 556 threshold; β_i is related to the dispersion of the lognormal cumulative function. In this

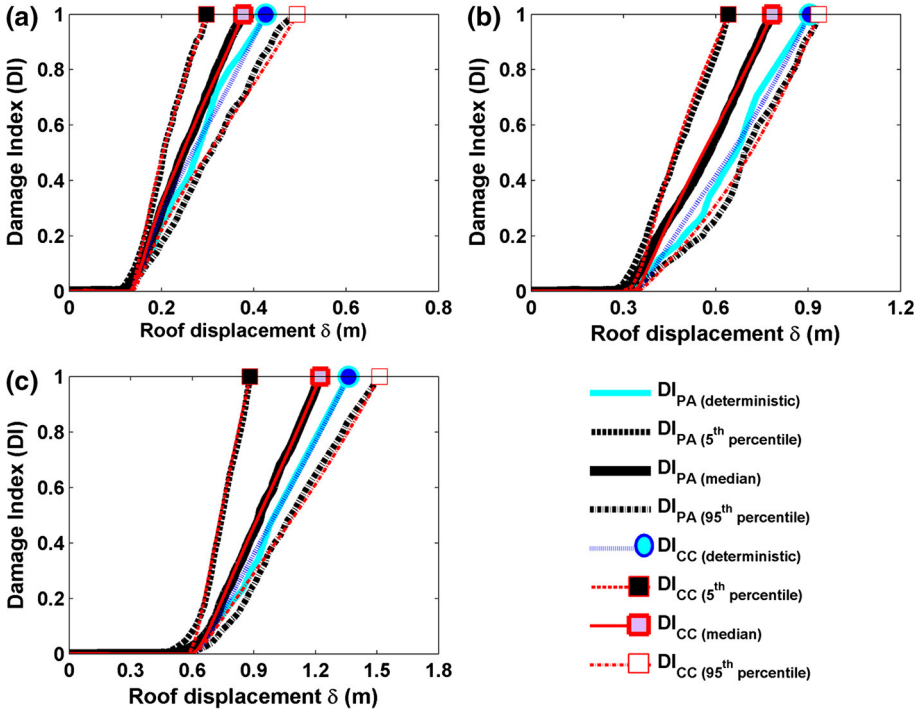


Fig. 15 DI_{CC} and DI_{PA} for the deterministic case and for the 5th, 50th and 95th percentiles. **a** SMF 3, **b** SMF 7 and **c** SMF 13

557 research four non-null damage states are considered: (1) *slight*, (2) *moderate*, (3) *severe*
 558 and (4) *complete*.

559 The main hypothesis underlying the construction of fragility curves herein are the
 560 following: (a) damage states thresholds, that is μ_i , are determined from capacity curve or
 561 from other criterion based, for instance, on observational data or expert opinion, and (b) the
 562 assumption that expected damage follows a binomial distribution (Grünthal 1998; Lago-
 563 marsino and Giovinazzi 2006) allows determining β_i . Be aware that the probability of
 564 exceedance in the damage state thresholds, μ_i , is 0.5; To decide the spectral displacements
 565 damage thresholds, two procedures are used here. The first one (Lagomarsino and Giovin-
 566 inazzi 2006) was proposed in the framework of the European Risk-UE project (see
 567 Milutinovic and Trendafiloski 2003) and is based on the bilinear form of the capacity
 568 curve, which is defined by the yielding point (S_{dy} , S_{ay}) and the ultimate capacity point
 569 (S_{du} , S_{au}). Thus, the Risk-UE based damage state thresholds are defined as follows:

$$\mu_1 = 0.7S_{dy}; \quad \mu_2 = S_{dy}; \quad \mu_3 = S_{dy} + 0.25(S_{du} - S_{dy}); \quad \mu_4 = S_{du} \quad (5)$$

571 The second one (Pujades et al. 2015) is based on DI_{PA} , or in its equivalent DI_{CC} , damage
 572 index. Spectral displacements corresponding to damage index (DI_{PA} or DI_{CC}) values of
 573 0.05, 0.2, 0.4, and 0.65, are allotted to the thresholds of the damage states *slight*, *moderate*,
 574 *severe*, and *complete*, respectively. Recall that these values are based on damage obser-
 575 vations (Park et al. 1985, 1987; Cosenza and Manfredi 2000). This way fragility curves for
 576 the four damage states are set up.
 577



Author Proof

578 Fragility curves easily allow us to obtain damage probability matrices (DPM), that is,
 579 the probability of the damage states $P_i(S_d)$. Then the mean damage state can be obtained,
 580 $D(S_d)$, also as a function of the same variable used to define fragility curves; often, the
 581 normalized mean damage state, $MDS(S_d)$ is used; how DPM, $D(S_d)$ and $MDS(S_d)$ are
 582 obtained from fragility curves is shown below. Once the fragility curves, $F_k(S_d)$, $k = 1, \dots,$
 583 4, are known, for each spectral displacement, S_d , $P_j(S_d)$, define the probability of the
 584 damage state j as a function of the spectral displacement, S_d . Equation (6) shows how these
 585 probabilities are obtained from fragility curves:

$$P_0(S_d) = 1 - F_1(S_d); \quad P_j(S_d) = F_j(S_d) - F_{j+1}(S_d) \quad j = 1 \dots 3 \quad P_4(S_d) = F_4(S_d) \quad (6)$$

587 P_0 corresponds to the probability of the null damage state, while indices 1–4 correspond
 589 to the four non-null damage states. Then the following equation defines the mean damage
 590 state $D(S_d)$ and the normalized mean damage state, $MDS(S_d)$:

$$D(S_d) = \sum_{j=0}^4 jP_j(S_d) = 4MDS(S_d) \quad (7)$$

593 As discussed in Pujades et al. (2015), MDS should not be compared directly with DI_{PA}
 594 because MDS has a statistical meaning and is based on the thresholds of the defined
 595 damage states, while DI_{PA} must be interpreted as a physical pointer, linked to the pro-
 596 gressive degradation of the bearing capacity of the building.

597 7.2 Results

598 Figure 16 shows the fragility curves, F_j , and the normalized mean damage state, MDS , as
 599 functions of spectral displacement. In this figure, the first row shows the case based on the
 600 Risk-UE project for the median capacity spectra shown in Fig. 11; the second row cor-
 601 responds to damage state thresholds based on the median DI_{PA} ; row 3 shows the case of the
 602 median DI_{CC} . Median DI_{PA} and DI_{CC} damage indices are shown in Fig. 15. Table 7 shows
 603 the parameters of these fragility curves (S_{d_i} , μ_i and β_i) for the deterministic and proba-
 604 bilistic 5th, 50th and 95th percentiles. The upper part of this table, corresponds to Risk-UE
 605 based fragility curves, in the middle the parameters corresponding to fragility curves based
 606 on DI_{PA} are given and the lower part shows the parameters of the fragility curves based on
 607 DI_{CC} . In fact, both indices are almost equivalent as DI_{PA} , has been used to fit DI_{CC} . Thus,
 608 the corresponding fragility and MDS functions are also similar. The μ_i and β_i values in the
 609 shadowy area correspond to the fragility curves of Fig. 16. Moreover, Fig. 17 compares the
 610 MDS functions, as defined by Eqs. (6) and (7), corresponding to these three cases. It can be
 611 seen that Risk-UE based MDS s overestimate the expected damage. However, in the case of
 612 the low-rise building SMF3, the Risk-UE based MDS underestimates the damage above
 613 the complete damage state. Disagreements between Risk-UE and DI_{PA} based MDS were
 614 also found by Pujades et al. (2015).



Author Proof

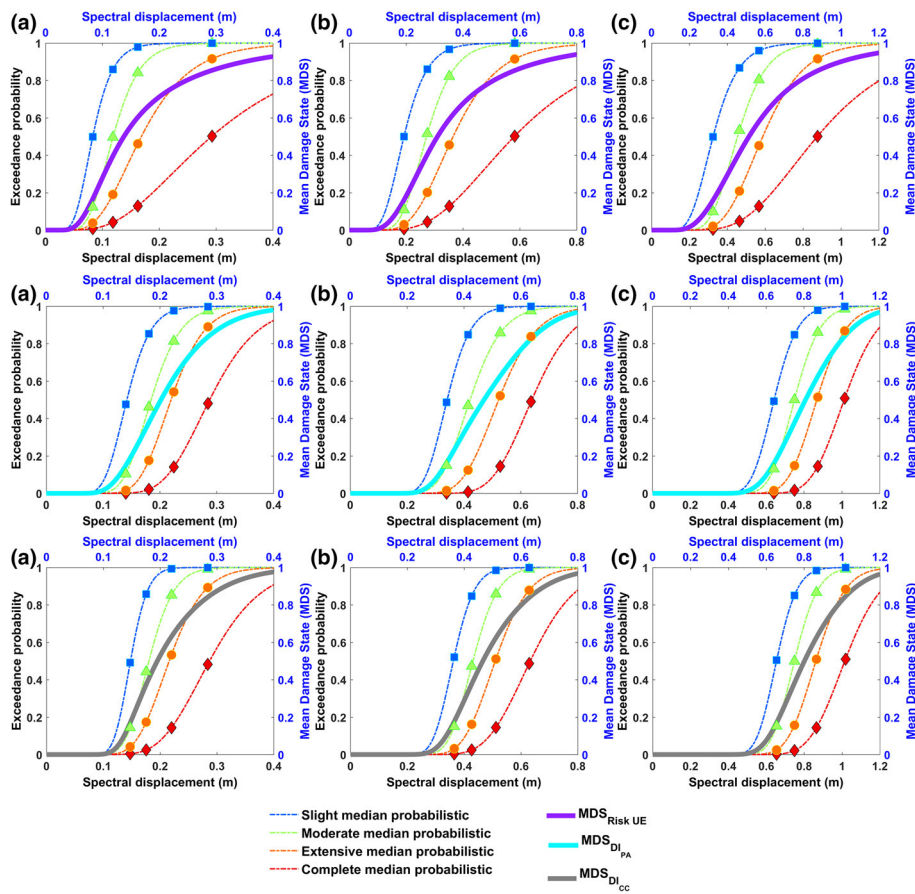


Fig. 16 Fragility curves and MDS functions obtained for median capacity spectra. Row 1 shows the case based on the risk-UE project, row 2 shows the case based on DI_{PA} , and row 3 shows the case based on DI_{CC}

8 Overview and discussion

8.1 Overview

In this paper, the parametric model for capacity curves and the new capacity-based damage index and fragility models, recently proposed by Pujades et al. (2015), have been tested and applied to steel buildings. High- (13 storeys), mid- (7 storey) and low-rise (3 storeys) buildings with special moment frames have been evaluated. Also, the seismic response of steel buildings, which are typical of the city of Mexico, has been investigated with deterministic and probabilistic approaches. NLSA and NLIDA are used. The probabilistic approach uses Monte Carlo simulation and optimization sampling techniques, such as the Latin hypercube technique. Uncertainties in the mechanical properties of buildings and in the seismic actions are considered. Only the strength and ductility of the structural elements are considered as random variables and it is assumed that they follow truncated normal or lognormal probability density distributions. For deterministic analyses, mean values of these distributions are used. Seismic actions are chosen according to the design

Table 7 Parameters of the fragility curves

	SMF3				SMF7				SMF13				
	ds ₁	ds ₂	ds ₃	ds ₄	ds ₁	ds ₂	ds ₃	ds ₄	ds ₁	ds ₂	ds ₃	ds ₄	
Risk-UE													
Deterministic													
μi (m)	0.084	0.122	0.179	0.329	0.194	0.277	0.389	0.668	0.335	0.470	0.623	0.971	
βi	0.340	0.340	0.440	0.570	0.330	0.310	0.390	0.510	0.320	0.270	0.320	0.420	
5th percentile													
μi (m)	0.080	0.112	0.148	0.231	0.187	0.260	0.329	0.474	0.311	0.434	0.502	0.629	
βi	0.320	0.270	0.320	0.420	0.310	0.250	0.260	0.340	0.290	0.220	0.210	0.210	
Median													
μi (m)	0.083	0.119	0.169	0.292	0.192	0.272	0.365	0.580	0.325	0.454	0.587	0.875	
βi	0.330	0.310	0.400	0.520	0.330	0.280	0.340	0.440	0.320	0.280	0.290	0.380	
95th percentile													
μi (m)	0.087	0.128	0.196	0.382	0.198	0.283	0.399	0.687	0.332	0.473	0.653	1.085	
βi	0.340	0.370	0.490	0.630	0.330	0.310	0.390	0.510	0.330	0.290	0.320	0.480	
D_{PA}													
Deterministic													
μi (m)	0.145	0.192	0.243	0.305	0.361	0.487	0.599	0.711	0.633	0.797	0.938	1.107	
βi	0.260	0.260	0.230	0.170	0.290	0.230	0.170	0.140	0.220	0.190	0.150	0.150	
5th percentile													
μi (m)	0.134	0.162	0.195	0.232	0.309	0.381	0.442	0.530	0.566	0.663	0.716	0.787	
βi	0.200	0.180	0.150	0.170	0.180	0.160	0.160	0.170	0.140	0.100	0.080	0.080	
Median													
μi (m)	0.142	0.185	0.220	0.288	0.341	0.422	0.522	0.639	0.643	0.750	0.868	1.012	
βi	0.230	0.220	0.210	0.230	0.190	0.210	0.200	0.180	0.150	0.140	0.140	0.140	



Table 7 continued

	SMF3				SMF7				SMF13			
	ds ₁	ds ₂	ds ₃	ds ₄	ds ₁	ds ₂	ds ₃	ds ₄	ds ₁	ds ₂	ds ₃	ds ₄
95th percentile												
μi (m)	0.150	0.208	0.269	0.352	0.382	0.545	0.663	0.766	0.705	0.862	1.001	1.202
βi	0.310	0.280	0.240	0.240	0.330	0.220	0.150	0.130	0.190	0.160	0.150	0.170
DI _{CC}												
Deterministic												
μi (m)	0.148	0.185	0.239	0.313	0.385	0.473	0.581	0.738	0.667	0.786	0.923	1.112
βi	0.190	0.220	0.260	0.270	0.180	0.200	0.210	0.210	0.150	0.150	0.160	0.170
5th percentile												
μi (m)	0.144	0.160	0.190	0.233	0.344	0.390	0.440	0.509	0.610	0.657	0.710	0.784
βi	0.120	0.130	0.160	0.200	0.110	0.120	0.130	0.140	0.070	0.070	0.080	0.090
Median												
μi (m)	0.148	0.180	0.216	0.287	0.362	0.431	0.510	0.633	0.654	0.749	0.862	1.015
βi	0.160	0.190	0.220	0.250	0.160	0.160	0.180	0.200	0.130	0.130	0.140	0.150
95th percentile												
μi (m)	0.153	0.195	0.259	0.359	0.399	0.516	0.645	0.790	0.668	0.822	0.993	1.237
βi	0.220	0.250	0.280	0.300	0.250	0.230	0.200	0.180	0.190	0.190	0.190	0.200

Upper rows correspond to Risk-UE based fragility curves, middle rows correspond to curves based on DI_{PA}, and lower rows correspond to fragility curves based on DI_{CC}. The deterministic case, and the 5th, 50th and 95th percentiles cases are shown. Median fragility curves are shown in Fig. 16

Non-null damage states: ds₁ (slight); ds₂ (moderate); ds₃ (extensive) and ds₄ (complete)

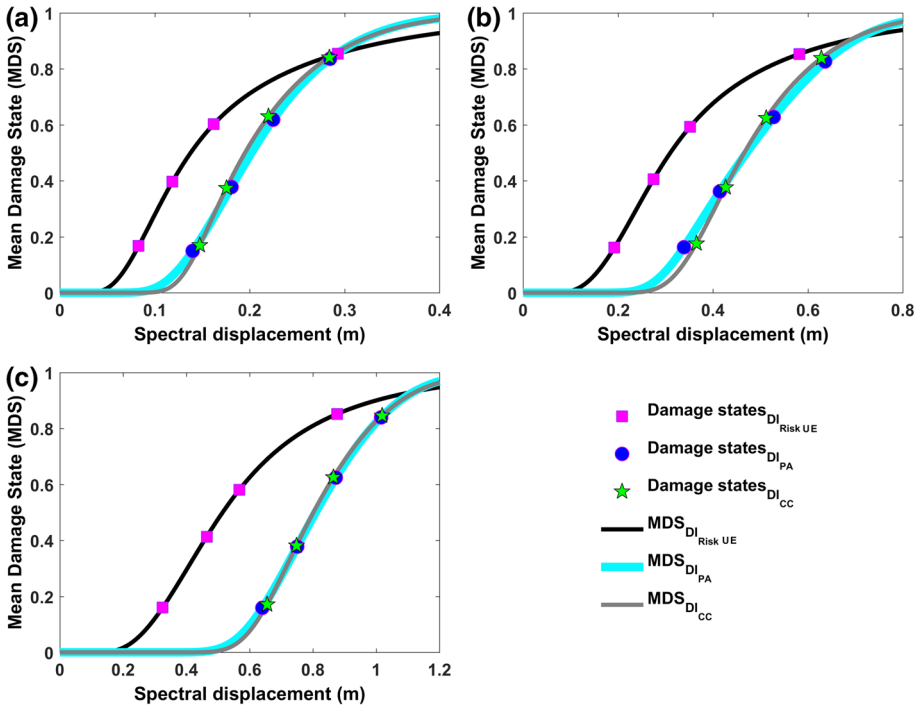


Fig. 17 Comparison of the median MDS functions. **a** SMF 3, **b** SMF 7 and **c** SMF 13

629 spectrum foreseen for soft soils in the city of Mexico. Thus, four accelerograms recorded in
 630 the study area have been selected, and a spectral matching technique has been applied, so
 631 that the response spectra match the design spectrum well. For deterministic analyses, the
 632 mean of these four matched accelerograms has been used. For probabilistic analysis, five
 633 probabilistic response spectra, with the design spectrum as mean and a predefined standard
 634 deviation have been generated. Then, for each generated spectrum, the spectral matching
 635 technique is applied to each of the four selected accelerograms, resulting in a suite of 20
 636 accelerograms, whose response spectra have the design spectrum as a mean and the pre-
 637 defined standard deviation.

638 8.2 Discussion

639 One of the main purposes of this research has been to check the parametric capacity model
 640 and the capacity-based damage index for steel buildings. Actually, Pujades et al. (2015)
 641 found a very simple analytical model with five independent parameters, fitting capacity
 642 curves well. It was shown how the degradation processes (damaging), which can be iso-
 643 lated in the nonlinear part of the capacity curve, are well represented by a cumulative
 644 integral of a cumulative lognormal function. That is by means of only two parameters. The
 645 appropriateness of the model may be clearly seen in the first derivatives of the capacity
 646 curves. Certainly, the use of a reinforced concrete building to illustrate the model was ad
 647 hoc because, at that moment, studies were being carried out on RC buildings. However, the
 648 parametric model wants to be valid for any capacity curve. Thus, this research highlights
 649 the validity of this fine model, also for steel buildings. Moreover, focusing on the nonlinear

part of the capacity curve, in the same paper, Pujades et al. (2015) proposed a new and simple damage index, which, like the Park and Ang damage index, is based on the stiffness degradation and on the energy loss. The parameter α is crucial as it separates the contribution of the stiffness degradation from that of the energy loss. Values around 0.7 are found for this parameter in the very few studies performed up to date. In fact, this value can be taken as a first quick estimate. Finer estimations require NLIDA. Results show that relatively low variations, around this value of 0.7, are expected, and they are related mainly with the characteristics of the seismic actions. This way, near-fault impulsive strong motions would lead to higher α values. Far-field seismic actions and soft soils would provide long duration seismic actions increasing the contributions to damage of repeated cyclic loads, thus decreasing the α values. Really, future research on more building types, using different seismic actions, can lead to tabulated values of this parameter, facilitating expedite and massive applications of this new damage index. Noticeably, also the fragility curves based on the damage thresholds defined according specific values of this damage index are dependent on the features of the seismic actions, which, on the other hand, would be reasonable.

Additional values of this research are the probabilistic approach adopted, as well as the study of the frame steel buildings located in soft soils of the Mexico City. Concerning the probabilistic approach, our results confirm that the probabilistic approach must be preferred because, due to the nonlinearity of the response of the buildings, the use of deterministic, even conservative, inputs, can lead to biased outputs; besides, probabilistic approach is richer as it allows obtaining and analyse the uncertainties in the response. Uncertainties in the response increase with the severity of the seismic actions. Concerning to the studied buildings, Fig. 18 shows PGA- δ and PGA-DI_{PA} curves obtained with NLIDA.

It can be seen how the high-rise frame steel buildings, located in soft soils in Mexico City, would exhibit no good performance, when subjected to likely seismic actions. Ongoing research (Díaz 2017) shows the adequacy of the use of protecting devices in those buildings. For instance, the use of Buckling Restrained Braced Frames, highly improves their seismic performance. Finally, the use of seismic actions recorded in the study zone, but that, at the same time, are compatible with the design response spectrum also gives reliability to the obtained results.

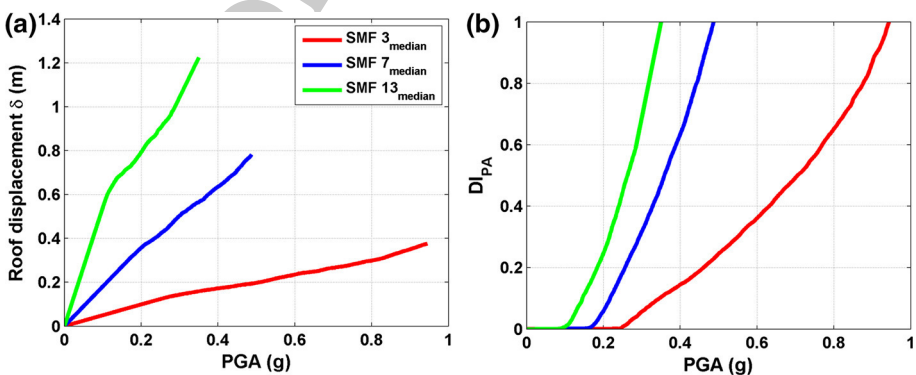


Fig. 18 PGA- δ and PGA-DI_{PA} functions for the three buildings (see explanation in the text)



682 9 Conclusions

683 Several relevant conclusions of this research are as follows:

- 684 • Because of nonlinearity both of static and dynamic responses, the use of mean, median
685 or characteristic values does not warranty to get mean, median or characteristic
686 responses. This fact highlights the importance of probabilistic approaches in front of
687 the more frequently used deterministic ones. Note that, in our case (see Fig. 13), the use
688 of mean values, both of the seismic actions and strength parameters, leads to un-
689 conservative results, which emphasizes, even more, the importance of probabilistic
690 approaches, which should be preferred, as they provide more complete, more valuable
691 and richer information.
- 692 • Uncertainties in the response increase with an increase in the severity of the
693 earthquake. The main source of uncertainty in the response is uncertainty in the seismic
694 action, but the influence of uncertainties in the mechanical parameters was also
695 significant, even though it was lower.
- 696 • The parametric model for capacity curves, the new damage index based on the secant
697 stiffness degradation and energy loss, and the corresponding fragility model as
698 proposed by Pujades et al. (2015) for reinforced concrete buildings, also provide
699 excellent results for the steel buildings studied herein. This confirms the robustness of
700 the parametric model, the compatibility of the new damage index with the Park and
701 Ang damage index, and the consistency of the fragility model with previous proposals
702 based on expert judgment.
- 703 • Concerning the damage index for the buildings and seismic actions studied in this
704 research, relative contributions to damage due to secant stiffness degradation and those
705 due to energy loss are respectively about 70 and 30%. The contribution to damage of
706 the energy loss is about 10% greater than that obtained by Pujades et al. (2015) for
707 reinforced concrete buildings. This increase is attributed to longer duration of the
708 accelerograms in Mexico City because of the combined effects of large epicentral
709 distances and soft soils. Longer durations entail greater numbers of hysteretic cycles for
710 the same spectral displacements, thus increasing the contribution to damage of energy
711 dissipation.
- 712 • For the steel buildings analysed here, static and dynamic analyses provide consistent
713 results. However, differences increase with the height of the buildings; this fact is
714 attributed to the influence of higher modes in the response, which in not captured in the
715 static analysis, as executed here.

716 The results of this research show that the parametric and damage models proposed by
717 Pujades et al. (2015) for reinforced 2D frame reinforced concrete buildings are also valid
718 for 2D frame steel buildings. Thus, this is a promising new tool that can be useful in rapid
719 damage assessments and, in particular, in probabilistic approaches, as it may allow sig-
720 nificant computation time reductions.

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