Probabilistic seismic assessment of multistory precast concrete frames exposed to corrosion

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Received: 3 December 2013 / Accepted: 6 April 2014 / Published online: 26 April 2014

1 Introduction

In current practice, durability design of concrete structures is primarily devoted to prevent and control local damage mechanisms of materials, such as corrosion of reinforcement and deterioration of concrete (Bertolini et al. 2004), usually without any quantitative assessment of their effects on the global structural performance. In fact, the check of durability

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requirements is limited to the control of the quality of materials and to the compliance of technical prescriptions, such as the minimum cover depth (CEB 1992). This approach does not consider the role of the structural system and does not account for the actual exposure conditions of the structural components. To overcome these limitations, the design of durable structures and the evaluation of the residual structural lifetime should refer to general methodologies capable to relate the time evolution of damage and structural performance not only to the properties of materials, but also to the quality of technical details, the structural typology and the exposure scenario, taking into account the uncertainties involved in the problem in a fully probabilistic framework (Biondini et al. 2004, 2006). These aspects assume a key role in the application of capacity design criteria for seismic design of concrete structures. In particular, damage induced by corrosion may lead over time to a significant progressive decay of strength and ductility of structural components, with a possible modification of the expected collapse mechanism and, consequently, of the energy dissipation capacity of the system (Biondini and Frangopol 2008; Biondini et al. 2011, 2013).

This paper focuses on the lifetime seismic assessment of precast structures, for which most of the structural members are often exposed to the atmosphere without any protection against environmental aggressiveness. Precast frame systems are characterized by dry connections with mechanical devices between beams and columns, typically designed to transfer shear forces only, without any dissipation capacity. Beam-to-column joints are usually hinged and the dissipative zones are located at the ends of the columns, where plastic hinges are expected to occur during a strong motion (Biondini and Toniolo 2009). For this type of systems, capacity design based on a collapse mechanism involving the maximum number of stories is needed to maximize the seismic capacity of the structure (Biondini et al. 2010). However, as aforementioned, seismic performance can change over time due to aging and progressive deterioration and a life-cycle design approach is necessary (Akiyama et al. 2011, 2012; Biondini et al. 2011, 2013).

For this purpose, a probabilistic approach for the lifetime assessment of seismic performance of concrete structures is presented in this paper by considering the interaction of seismic and environmental hazards (Biondini et al. 2011, 2013). In the proposed approach the corrosion of reinforcement due to chloride diffusion is considered, since contamination by chlorides is one of the most significant sources of environmental hazard for concrete structures (Stewart and Rosowsky 1998). Chloride diffusion and the evolution of deterioration is described using cellular automata (Biondini et al. 2004). Structural modeling is based on beam finite elements with material nonlinearity lumped at the beam ends (Ibarra et al. 2005; Haselton 2006). The seismic capacity is evaluated through nonlinear static and dynamic analyses (Fajfar 1999; Vamvatsikos and Cornell 2002). The uncertainties involved in the problem are taken into account in probabilistic terms (Ang and Tang 2007) through a Monte Carlo simulation with stratified sampling (Iman and Conover 1982; Stein 1987; Helton and Davis 2003).

The proposed methodology is applied for the probabilistic lifetime seismic assessment of multistory precast buildings (Titi 2012). The results show that structures designed for the same seismic action could have different lifetime seismic performance depending on the environmental exposure. This demonstrates the need for a life-cycle approach to both seismic assessment of existing buildings and seismic design of new structures, and indicates that capacity design criteria need to be properly revised to consider the severity of the environmental exposure and the evolution of damage over the structural lifetime.

2 Evaluation of seismic performance

2.1 Structural modeling

The structural modeling for the assessment of seismic performance is based on beam finite elements with material nonlinearity lumped at the beam ends, where plastic hinges are expected to occur. The nonlinear behavior of the plastic hinges is defined in terms of bending moment M versus rotation θ relationship. The cyclic model proposed by Ibarra et al. (2005) is adopted, with the trilinear envelope curve shown in Fig. 1. The bending moment at yielding, M_y, corresponds to the first yielding of steel reinforcement and is evaluated by taking the effects of concrete confinement into account (Mander et al. 1988). The other parameters, i.e. the initial stiffness K_e, the capping to yielding bending moment ratio M_c/M_y, the capping rotation θ_c, and the post-capping rotation θ_{pc}, are evaluated using the calibration equations proposed by Haselton (2006). The hysteretic model includes multiple modes of cyclic damage leading to collapse of concrete frames (Fischinger et al. 2008). In particular, with reference to Fig. 2, cyclic damage at step i is applied to yielding strength M_i, capping strength defined by the bending moment $M_{ref,i}$ evaluated at the intersection of the M-axis with the projection of the post-capping branch, hardening stiffness $K_{h,i}$, unloading stiffness $K_{u,i}$, and reloading stiffness defined in terms of target rotation $\Delta\theta_{t,i}$:

$$M_i^{+/-} = (1-\beta_i)\, M_{i-1}^{+/-} \tag{1}$$

$$M_{ref,i}^{+/-} = (1-\beta_i)\, M_{ref,i-1}^{+/-} \tag{2}$$

$$K_{h,i}^{+/-} = (1-\beta_i)\, K_{h,i-1}^{+/-} \tag{3}$$

$$K_{u,i} = (1-\beta_i)\, K_{u,i-1} \tag{4}$$

$$\Delta\theta_{t,i}^{+/-} = (1+\beta_i)\, \Delta\theta_{t,i-1}^{+/-} \tag{5}$$

where β_i is a damage energy-based index defined as follows:

$$\beta_i = \left(\frac{E_i}{E_t - \sum_{j=1}^{i} E_j}\right)^c \tag{6}$$

where E_i is the hysteretic energy dissipated at step i, $E_t = 2\lambda E_y$ is the reference hysteretic energy dissipation capacity expressed as a fraction λ of twice the elastic strain energy at yielding $2E_y = M_y\theta_y$, and c is an exponent related to the rate of cyclic damage ($c = 1$ is assumed). The parameter λ is evaluated based on experimental results as discussed in Haselton (2006). Throughout the loading history β_i has to be in the interval [0,1]. When

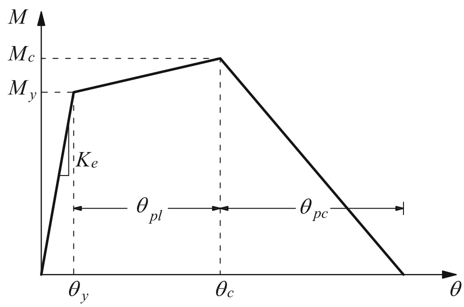


Fig. 1 Bending moment M versus rotation θ envelope curve of the hysteretic model

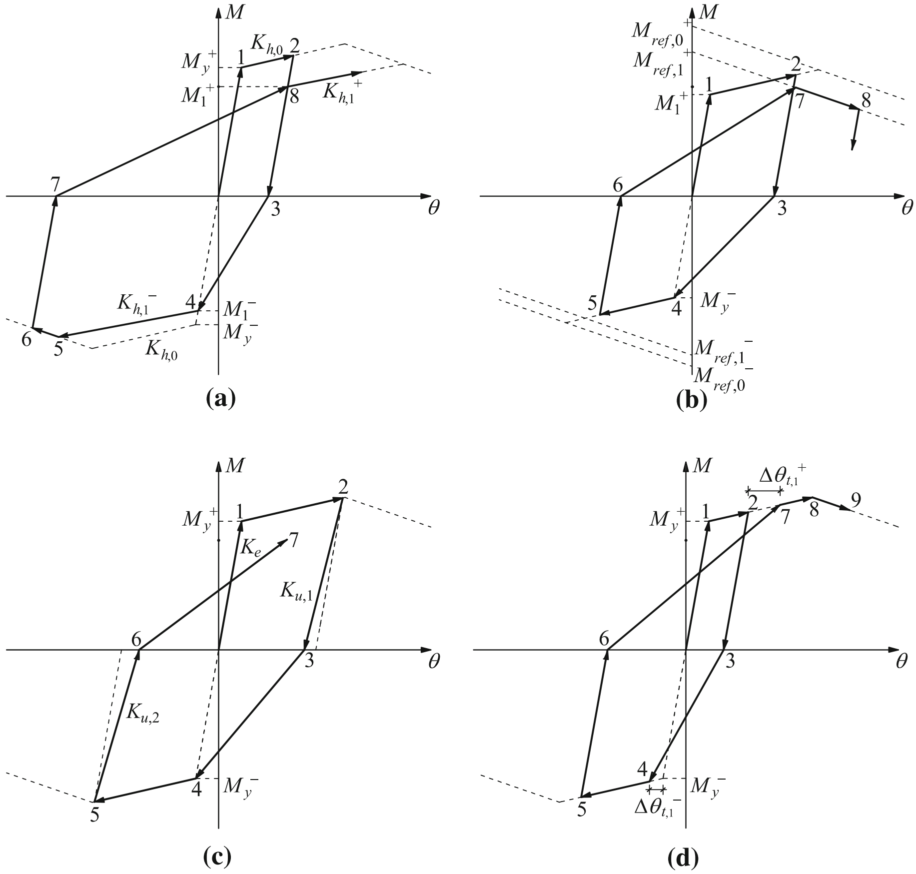


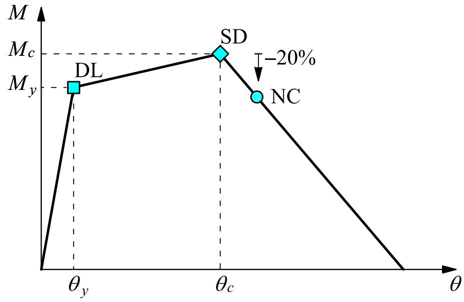
Fig. 2 Hysteretic model: **a** basic strength deterioration; **b** post-capping strength deterioration; **c** unloading stiffness deterioration; **d** reloading stiffness deterioration (Ibarra et al. 2005)

these limits are exceeded, hysteretic energy capacity drops to zero and collapse is assumed to take place (Ibarra et al. 2005).

2.2 Nonlinear static and dynamic analysis methods

In this study, the seismic capacity of precast buildings is evaluated by means of nonlinear static and dynamic analysis procedures. Nonlinear static analysis is based on the N2 method (Fajfar 1999). Nonlinear dynamic analysis under seismic loading is performed by increasing step by step the seismic intensity up to collapse (Vamvatsikos and Cornell 2002). The aim of the incremental dynamic analysis (IDA) is to relate an intensity measure (IM) to a damage measure (DM). Typical IMs are the peak ground acceleration (PGA) and the 5 % damped spectral acceleration associated to the first mode period of the structure. In this study the PGA is assumed as IM, since for flexible precast systems the higher vibration modes can provide significant contributions. Several quantities, usually related to displacements or dissipated energy, can be assumed as DMs. For structural damage of multistory frame buildings, a DM closely related to both local and global damage is the maximum interstory drift $\theta_{\max} =$

Fig. 3 Achievement of limit states on the envelope curve of the hysteretic model



Δ_{max}/h, where Δ_{max} is the maximum interstory displacement and h is the interstory height (Vamvatsikos and Cornell 2002).

2.3 Definition of limit states

Seismic capacity is evaluated with reference to the following limit states:

- DL: Damage Limitation limit state;
- SD: Significant Damage limit state;
- NC: Non Collapse limit state.

The attainment of such limit states can be related to the local behavior of the plastic hinges (Dolšek 2011). The following limit values of the rotation θ are assumed (Fig. 3):

- yielding rotation $\theta = \theta_y = M_y/K_e$;
- capping rotation $\theta = \theta_c$;
- ultimate rotation $\theta_u > \theta_c$ corresponding to a bending moment $M_u = 0.8M_c$.

The DL limit state is attained when all columns at any story of the concrete frame reach the yielding rotation θ_y. The SD and NC limit states are attained when one of the columns reaches the capping rotation θ_c and the ultimate rotation θ_u, respectively. However, if the NC base shear is lower than 80 % of the maximum base shear capacity, such limits are relaxed. In this case the NC limit state is related to a post-peak drop of 20 % of the base shear with respect to the maximum base shear capacity, and the SD limit state is defined at the 75 % of the NC top displacement (Dolšek 2011).

This definition of the limit states is recommended when the structural performance is investigated by means of nonlinear static analysis, since the rotation θ represents an effective indicator of local damage under monotonic behavior. However, for structures under cyclic loading the limit states can be more effectively related to global quantities, such as the maximum interstory drift θ_{max} (Building Seismic Safety Council 2000):

- DL: $\theta_{max} = 1\,\%$;
- SD: $\theta_{max} = 2\,\%$;
- NC: $\theta_{max} = 4\,\%$.

This approach is generally adopted when the structural performance is investigated by means of incremental dynamic analysis with DM $= \theta_{max}$.

The results of nonlinear static analyses and time-history analyses of multistory concrete frames show a good correspondence of the limit states defined in terms of column rotation and interstory drift (Titi 2012).

3 Structural deterioration

Lifetime seismic assessment of concrete structures in an aggressive environment should account for both the diffusion process of the aggressive agents, such as chlorides, and the mechanical damage induced by diffusion, which usually involves corrosion of reinforcement (Bertolini et al. 2004).

3.1 Diffusion process

The diffusion process is described by the Fick's laws which, for single component diffusion in isotropic, homogeneous and time-invariant media, can be reduced to the following second order partial differential linear equation (Glicksman 2000):

$$\frac{\partial C}{\partial t} = D\nabla^2 C \tag{7}$$

where D is the diffusivity coefficient of the medium, $C = C(\mathbf{x}, t)$ is the concentration of the chemical component at point $\mathbf{x}$ and time t, $\nabla C = \mathbf{grad}\, C(\mathbf{x}, t)$ and $\nabla^2 = \nabla \cdot \nabla$.

For one-dimensional diffusion (1D) the Fick's laws can be solved analytically. The Fick's 1D model is therefore a convenient mathematical tool for practical applications (fib 2006). However, the actual diffusion process in concrete structures is generally characterized by two- or three-dimensional patterns of concentration gradients, and simplified 1D diffusion models can lead to a significant loss of accuracy depending on the exposure conditions, geometrical shape ratio of the cross-section, and location of points where the concentration is evaluated (Titi and Biondini 2012). An effective numerical solution of the general 2D and 3D Fick's diffusion differential equations is achieved by means of cellular automata (Biondini et al. 2004, 2006; Biondini 2011).

3.2 Damage index

Damage induced by diffusion is studied by considering the effects of uniform corrosion in terms of mass loss of the reinforcing steel bars. The percentage loss of steel resistant area for a corroded steel bar is described by means of a dimensionless damage index $\delta_s = \delta_s(t)$ which provides a direct measure of damage within the range [0,1]. In this way, the area A_s of the corroded steel bar can be represented as a function of the damage index as follows:

$$A_s(t) = [1 - \delta_s(t) A_{s0}] \tag{8}$$

where A_{s0} is the area of the undamaged bar.

The corrosion rate depends on the concentration $C = C(\mathbf{x}, t)$ of the chemical substance. Such a process involves several factors, including temperature and humidity, and its dynamics are governed by coupled diffusion of heat, moisture and various chemical substances (Saetta et al. 1993; Xi and Bažant 1999; Xi et al. 2000). Moreover, the available information about environmental agents and material characteristics is usually not sufficient for a detailed modeling. Despite the complexity of the problem, simple degradation models can be often successfully adopted. Based on available data for sulphate and chloride attacks (Pastore and Pedeferri 1994), the following linear dependency is assumed (Biondini et al. 2004):

$$\frac{\partial \delta_s}{\partial t} = \rho C(\mathbf{x}, t) \tag{9}$$

where ρ is a steel damage rate coefficient.

Table 1 Probability distribution and coefficients of variation (mean values = nominal values)

Random variable ($t = 0$)	Distribution	C.o.V.
Mass, m	Normal $^{(*)}$	0.10
Concrete strength, f_c	Lognormal	5 MPa/$f_{c,mean}$
Steel strength, f_{sy}	Lognormal	30 MPa/$f_{sy,mean}$
Viscous damping, ξ	Normal $^{(*)}$	0.40
Initial stiffness, K_e	Lognormal	0.28
Capping to yielding bending moment ratio, M_c/M_y	Lognormal	0.10
Capping rotation, θ_c	Lognormal	0.45
Post-capping rotation, θ_{pc}	Lognormal	0.72
Energy dissipation capacity, λ	Lognormal	0.49

* Truncated distribution with non-negative outcomes

4 Probabilistic modeling

The time-variant seismic capacity of structural systems is affected by uncertainty and has to be investigated in probabilistic terms. To fulfil this goal, a lifetime probabilistic analysis is carried out by using Monte Carlo simulation (Ang and Tang 2007) based on a probabilistic modeling of the diffusion process and damage propagation of concrete members exposed to corrosion (Biondini et al. 2006).

The probabilistic model assumes the following quantities as random variables: mass m at each story; concrete compression strength f_c; steel yielding strength f_{sy}; viscous damping ξ; initial stiffness K_e; ratio of capping moment to yielding moment M_c/M_y; rotation at capping point θ_c; rotation of the post-capping branch θ_{pc} (Fig. 1); energy dissipation capacity λ (Ibarra et al. 2005). These random variables are assumed uncorrelated with mean value equal to the nominal value. The probabilistic distribution and coefficients of variation of the random variables are listed in Table 1 (Ellingwood 1980; Porter et al. 2002; Biondini et al. 2006; Haselton 2006; Dolšek 2009).

In order to reduce the computational cost, the Monte Carlo simulation is performed adopting a Latin Hypercube Sampling (LHS). LHS uses the stratification of the theoretical probability functions of the random variables to reduce the size of sample data. Two steps are required in order to build the sampling matrix: (1) samples from marginal distributions are chosen according to the probability density functions, and (2) the sample matrix is re-arranged in order to match within a certain tolerance a target correlation matrix (Iman and Conover 1982). The Simulated Annealing method is used to minimize an error norm of the actual correlation matrix with respect to the target matrix (Kirkpatrick et al. 1983; Vořechovský and Novák 2003, 2009).

5 Applications

5.1 Multistory precast frames

The multistory precast concrete frames with pinned beam-to-column joints shown in Fig. 4 are considered. With reference to the column cross-sections shown in Fig. 5, the seven frame systems identified in Table 2 are studied. The nominal effective weight at each story is

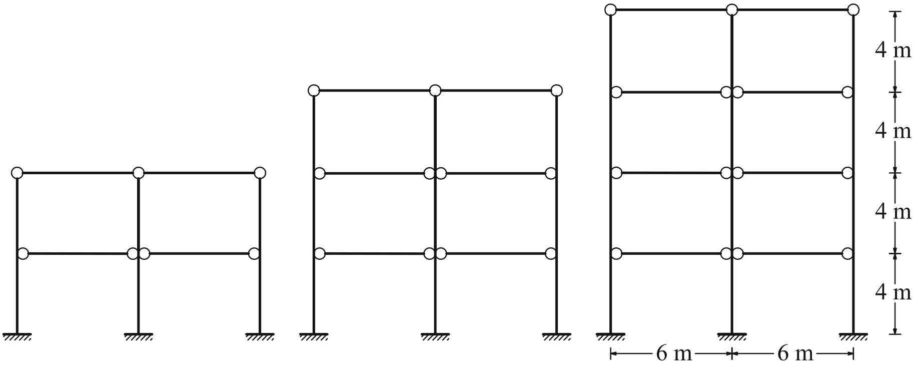


Fig. 4 Multistory precast concrete frames with pinned beam-to-column joints

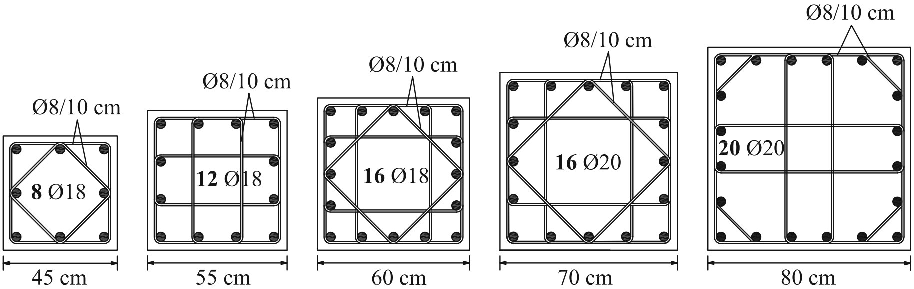


Fig. 5 Column cross-sections of the multistory precast concrete frames

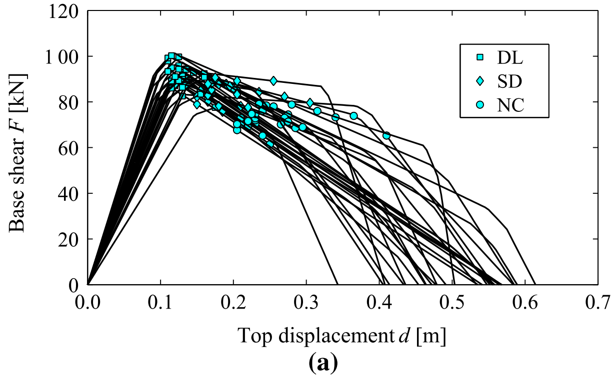
Table 2 Number of stories and column cross-sections of the multistory frames (Figs. 4, 5)

No. of stories n	2				3		4
Side b [cm]	45	55	70	80	60	80	80
No. of steel bars and bar diameter Φ [mm]	8 Φ18	12 Φ18	16 Φ20	20 Φ20	16 Φ18	20 Φ20	20 Φ20

$w = mg = 1{,}200\,\text{kN}$. A nominal value of the viscous damping $\xi = 3\,\%$ is assumed. The nominal material strengths of concrete in compression and reinforcing steel are $f_c = 48\,\text{MPa}$ and $f_{sy} = 450\,\text{MPa}$, respectively. The ultimate bending curvature of the cross-sections is computed by assuming the strain limits $\varepsilon_{cu} = 0.35\,\%$ and $\varepsilon_{su} = 7.5\,\%$ for unconfined concrete in compression and reinforcing steel, respectively. The effects of concrete confinement are taken into account as proposed by Mander et al. (1988). Shear failure is avoided based on a proper capacity design (Biondini et al. 2010). The columns are modeled using beam elements with lumped plasticity (Mazzoni et al. 2006). The parameters of the envelope curve of the cyclic model, shown in Fig. 1, are computed using the calibration equations proposed by Haselton (2006). Elastic behavior is assumed for the beams.

Table 3 Error norm e of the correlation matrix for different values of the sample size N

Sample size N	10	15	20	30	50
Error norm e	0.0057	0.0021	0.0010	0.0005	0.0005



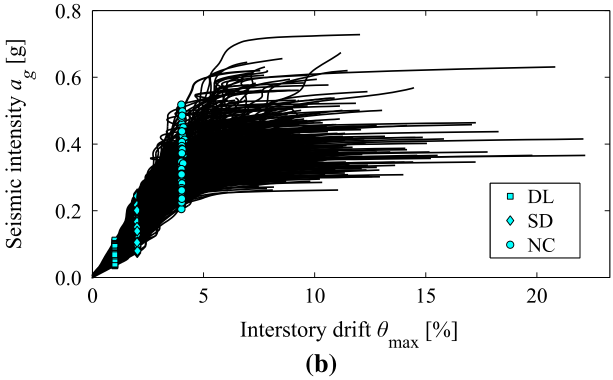


Fig. 6 Seismic performance of the two-story frame with column cross-section 45 cm × 45 cm: **a** capacity curves base shear—top displacement and **b** IDA curves seismic intensity—interstory drift with indication of the attainment of the limit states DL, SD and NC (sample size $N = 30$)

5.2 Probabilistic assessment of seismic performance

The seismic performance of the frames is evaluated in probabilistic terms by means of nonlinear static analyses (N2) and nonlinear dynamic analyses (IDA). Recorded or artificial accelerograms can be used to perform time-history analyses. Recorded accelerograms reproduce real earthquakes, but large samples are generally necessary to reduce the spectral mismatch of the individual accelerograms with respect to the response spectra provided by the seismic codes. Artificial accelerograms generated to match with good accuracy target response spectra (SIMQKE 1976) are often more suitable for probabilistic investigations (Biondini and Toniolo 2009).

The probabilistic analysis is performed by Monte Carlo simulation according to the model given in Table 1. The expected spatial variability of geometrical and mechanical quantities is limited. Therefore, full correlation is assumed for the sets of random variables associated

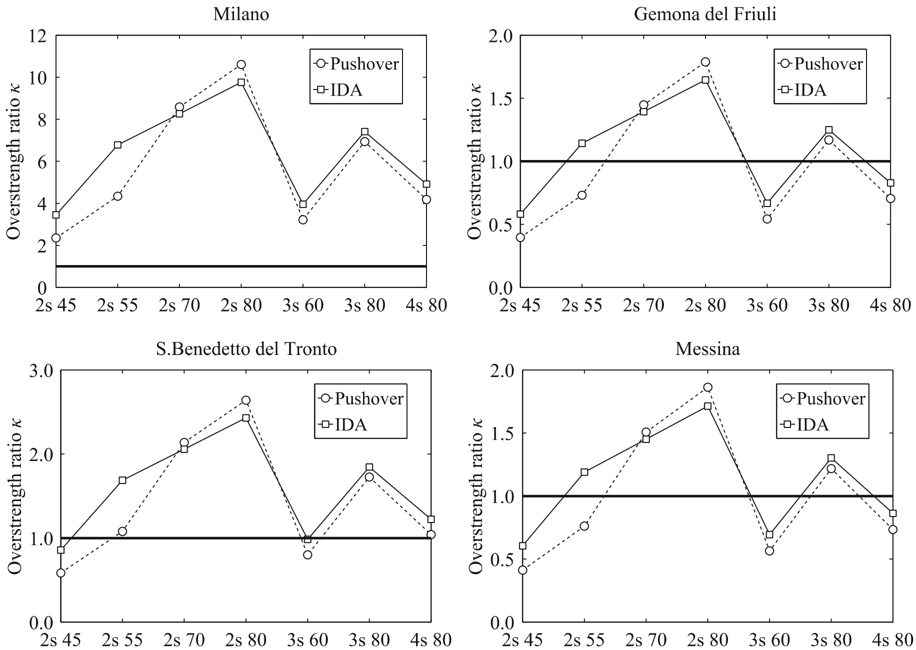


Fig. 7 Overstrength ratio $\kappa = a_g/a_{gd}$ of the 5 % fractile of the seismic capacity a_g with respect to the design value a_{gd} of the corresponding seismic demand evaluated for four sites in Italy with different levels of seismicity (Milano, Gemona del Friuli, S. Benedetto del Tronto, Messina). Label "*n*s *b*" = *n*-story frame with column size $b \times b$ (cm) (see Table 2)

to the columns. For each realization of the simulation process, IDA analyses are performed for a set of 25 artificial accelerograms compatible with the elastic response spectrum given by Eurocode 8 for soil type B (CEN-EN 1998–1 2004). The error norm e between the actual correlation matrix and the target matrix is provided in Table 3 for different values of the sample size N. It is found that a sample size $N = 30$ provides good accuracy in terms of statistical parameters (Dolšek 2009).

Figure 6 shows for a sample size $N = 30$ the set of capacity curves (Fig. 6a) and IDA curves (Fig. 6b) with indication of the attainment of the limit states DL, SD and NC for the two-story building with column cross-section 45 cm × 45 cm. A comparison between the results of nonlinear static analysis and nonlinear dynamic analysis is given in Fig. 7 for the NC limit state in terms of overstrength ratio $\kappa = a_g/a_{gd}$ of the 5 % fractile of the seismic capacity a_g (Kramar et al. 2010) with respect to the design value a_{gd} of the corresponding seismic demand evaluated for four sites in Italy characterized by different levels of seismicity (Milano, Gemona del Friuli, S. Benedetto del Tronto, Messina). The comparison indicates that nonlinear static analysis and nonlinear dynamic analysis provide similar results.

5.3 Time evolution of seismic performance

The precast concrete frames are subjected to a diffusive attack from chlorides located on the external surfaces of the columns. The exposure scenario is shown in Fig. 8, with chloride concentration $C_0 = 3\,\%$ (wt.%/c) on the outermost side of the two lateral columns and αC_0 on the other column sides. A nominal diffusivity coefficient $D = 10^{-11}\ \mathrm{m}^2/\mathrm{s}$ is assumed. Figure 9 shows the maps of concentration $C(\mathbf{x}, t)/C_0$ of the cross-section 45 cm × 45 cm

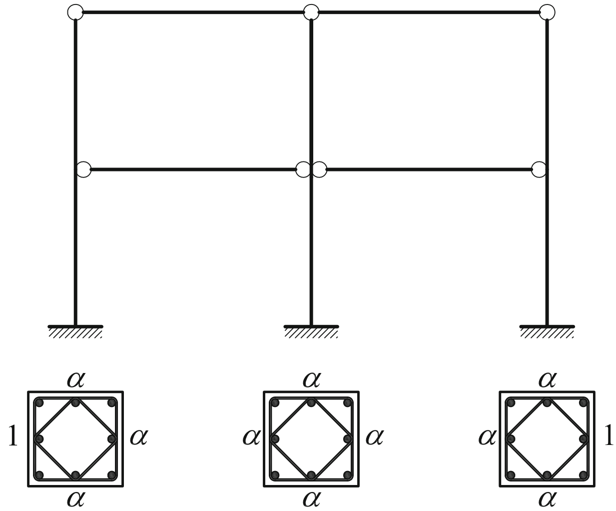


Fig. 8 Damage scenario and exposure levels $0 \leq \alpha \leq 1$

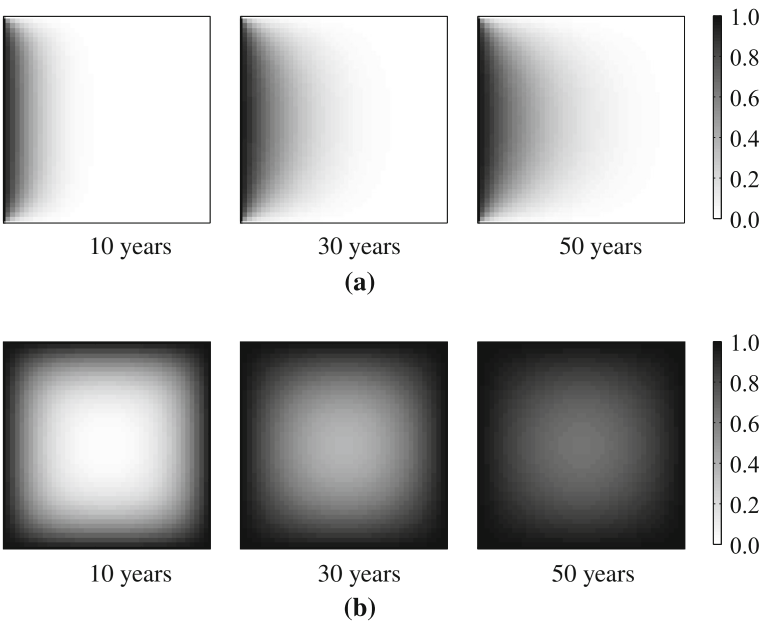


Fig. 9 Maps of concentration $C(\mathbf{x}, t)/C_0$ of the aggressive agent for a lateral column after 10, 30 and 50 years from the initial time of diffusion penetration: **a** $\alpha = 0$; **b** $\alpha = 1$ (see Fig. 8)

for two exposure levels, $\alpha = 0$ and $\alpha = 1$, after 10, 30, and 50 years from the initial time of diffusion penetration.

The corrosion damage induced by diffusion is evaluated by assuming a nominal damage rate coefficient $\rho C_0 = 0.02\,\mathrm{s}^{-1}$, with corrosion initiation associated to a critical threshold of concentration $C_{cr} = 0.6\,\%$ (wt.%/c) (fib 2006). This damage propagation model involves a severe corrosion of steel reinforcement, as may occur for carbonated or heavily chloride-contaminated concrete and high relative humidity (Bertolini et al. 2004). The corresponding

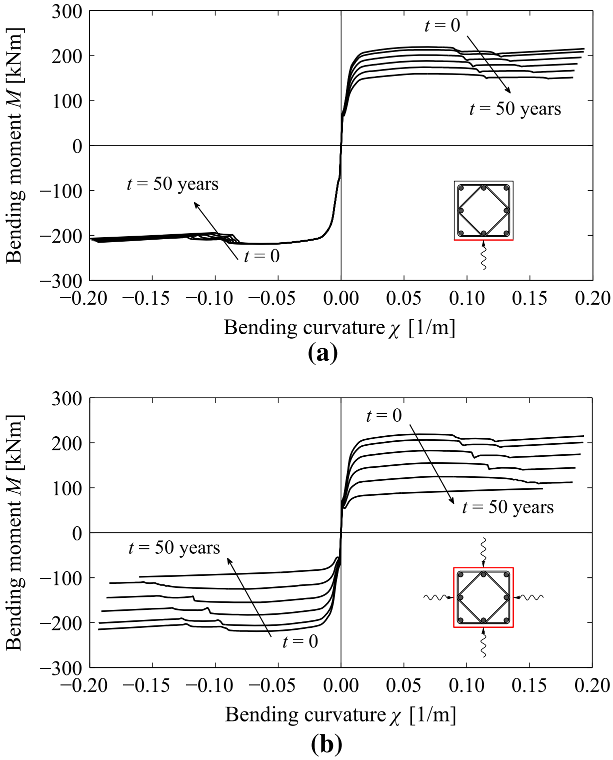


Fig. 10 Time evolution of the nominal bending moment—curvature diagram of the cross-section with size 45 cm $\times$ 45 cm at the base of a lateral column of the two-story building for two exposure levels **a** $\alpha = 0$ and **b** $\alpha = 1$ ($\Delta t = 10$ years)

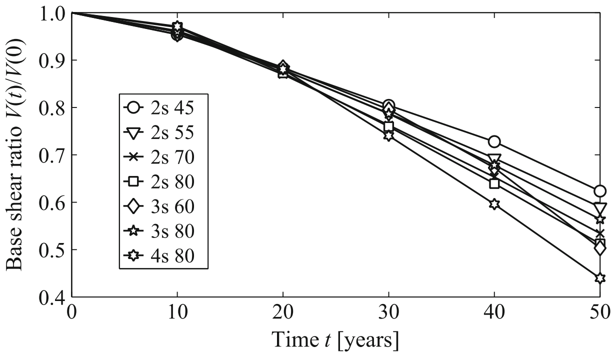


Fig. 11 Time evolution of the mean value of the base shear capacity $V(t)$ of the frames with respect to the initial value $V(0)$ for the NC limit state and the most severe exposure ($\alpha = 1$). Label "ns b" = n-story frame with column size $b \times b$ (cm) (see Table 2)

time evolution of the nominal bending moment—curvature diagram of the cross-section with size 45 cm $\times$ 45 cm at the base of a lateral column of the two-story building is shown in Fig. 10 for two exposure levels, $\alpha = 0$ and $\alpha = 1$.

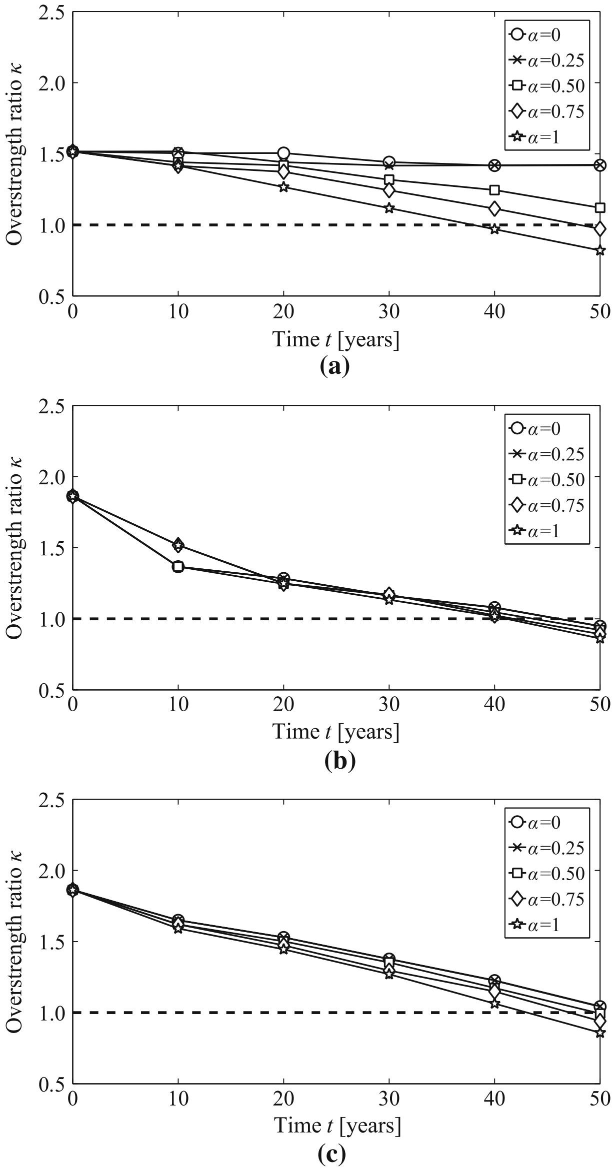


Fig. 12 Time evolution of 5 % fractile of the overstrength ratio κ associated to the limit states **a** DL, **b** SD and **c** NC of the two-story frame with column cross-section 80 cm $\times$ 80 cm for the site of Messina and different levels of exposure ($\alpha = 0, 0.25, 0.50, 0.75, 1$)

The probabilistic time evolution of seismic performance is studied through nonlinear static analyses, due to their computational efficiency. The lifetime probabilistic assessment of structural performance of the frames is shown in Fig. 11, which provides the mean value of the base shear capacity at the NC limit state, normalized with respect to the initial value, for the most severe exposure scenario ($\alpha = 1$). It can be noticed that in such conditions the load-bearing capacity can have in 50 years a significant reduction (over 50 %) with respect

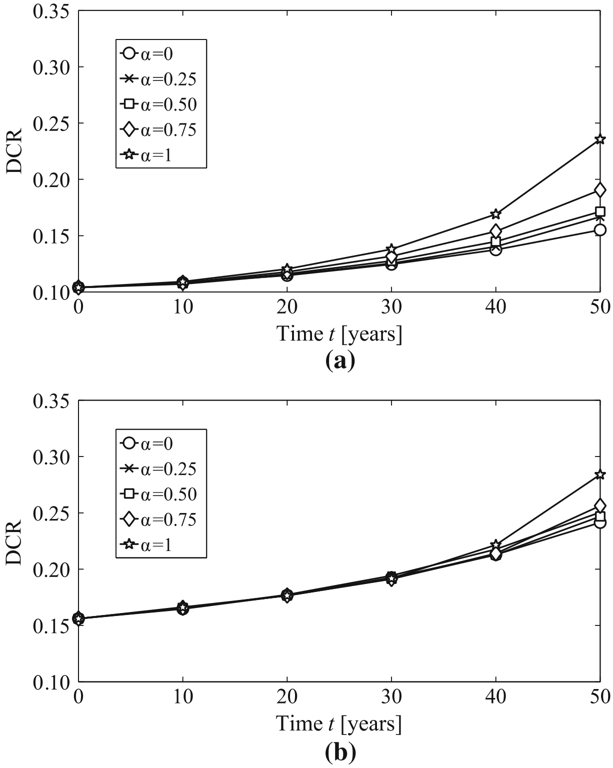


Fig. 13 Time evolution of the shear demand—capacity ratio (DCR) at the base of the two-story buildings with column cross-section **a** 45 cm × 45 cm and **b** 80 cm × 80 cm, for different levels of exposure ($\alpha = 0, 0.25, 0.50, 0.75, 1$)

to the initial value of the undamaged structure. Therefore, depending on the level of seismic intensity and the exposure scenario, over time the seismic capacity may be not compatible with the corresponding demand. As an example, Fig. 12 shows the 5 % fractile of the overstrength ratio κ associated to the limit states DL, SD and NC of the two-story frame with column cross-section 80 cm × 80 cm for the site of Messina and different levels of environmental exposure. It can be noticed that the seismic capacity decreases over time and the overstrength reaches in some cases values $\kappa < 1$.

The reduction over time of the seismic performance can be associated to a reduction of the base shear capacity, as well as to a different location of the flexural plastic hinges which may involve a consequent change of the expected collapse mechanism and dissipation capacity of the system (Biondini and Frangopol 2008; Biondini et al. 2011, 2013). It is worth noting that changes in the collapse mechanism may be also due to corrosion of transversal steel reinforcement. In particular, a reduction of shear strength of the columns can modify the capacity design and lead to brittle shear failures. However, for the frame systems investigated in this study such kind of failure can be excluded by checking the time-variant story shear demand—capacity ratio (DCR) defined as:

$$DCR = \frac{V_{Sd}}{V_{Rd}} \tag{10}$$

where V_{Sd} is the shear demand at each story and V_{Rd} is the sum of the shear strength of the columns at the same story, evaluated according to Eurocode 8 (CEN-EN 1998–1 2004). As an example, Fig. 13 shows the time evolution of the DCR for the columns at the base of the two-story buildings with cross-section 45 cm $\times$ 45 cm (Fig. 13a) and 80 cm $\times$ 80 cm (Fig. 13b). The limit condition DCR < 0.5 recommended by Celarec et al. (2011) to avoid potential brittle failure in shear is satisfied over the structural lifetime.

6 Conclusions

Seismic design codes are currently based on time-invariant capacity design criteria and do not take into account the interaction with environmental hazards. However, a lifetime approach is necessary for the seismic design of deteriorating structures, since undesired collapse mechanisms, which are thought to be avoided at the design stage, can arise over time and significantly modify the seismic performance. Moreover, due to the uncertainties involved in the problem, a measure of the time-variant seismic performance is realistically possible only in probabilistic terms.

Based on these considerations, a probabilistic methodology for the lifetime assessment of seismic performance of concrete structures exposed to corrosion has been presented. The proposed approach included an effective tool to model the deterioration process into reinforced concrete members at cross-sectional level and, hence, to evaluate their lifetime performance in terms of bending moment—curvature diagrams. The global effects of the local damage phenomena on the overall seismic performance have been investigated with reference to precast structures, for which most of their structural members are usually exposed to the atmosphere without any protection against environmental aggressiveness.

The results of nonlinear static and dynamic analyses showed that the variation over time of the column flexural strength and ductility could significantly affect the structural response of multistory precast frames under seismic excitation depending on the environmental exposure. Therefore, these results highlighted the need of a proper evaluation of the combined effects of seismic damage and deterioration processes and emphasized the importance of a life-cycle approach for the seismic design of resilient concrete structures exposed to environmental hazard (Titi and Biondini 2013). Under this perspective, time-variant capacity design criteria taking the severity of the environmental aggressiveness into account are necessary. This aim can be achieved by relating both force reduction factors (behavior factors) at the system level, and over-strength factors at the member level, to the environmental exposure of the structure.

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