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ORIGINAL RESEARCH PAPER

Building design based on energy dissipation: a critical assessment

Chara Ch. Mitropoulou · Nikos D. Lagaros · Manolis Papadrakakis

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Abstract The basic objective of this study is the assessment of the European seismic design codes and in particular of EC2 and EC8 with respect to the recommended behaviour factor q. The assessment is performed on two reinforced concrete multi-storey buildings, having symmetrical and non-symmetrical plan view respectively, which were optimally designed under four different values of the behaviour factor. In the mathematical formulation of the optimization problem the initial construction cost is considered as the objective function to be minimized while the cross sections and steel reinforcement of the beams and the columns constitute the design variables. The provisions of Eurocodes 2 and 8 are imposed as constraints to the optimization problem. Life-cycle cost analysis, in conjunction with structural optimization, is believed to be a reliable procedure for assessing the performance of structures during their life time. The two most important findings that can be deduced are summarized as follows: (1) The proposed Eurocode behaviour factor does not lead to a more economical design with respect to the total life-cycle cost compared to other values of q ($q = 1, 2$). (2) The differences of the total life-cycle cost values may be substantially greater than those observed for the initial construction cost for four different q ($q = 1, 2, 3, 4$).

Keywords Performance-based design · Behaviour factor · Life-cyclecost analysis · RC buildings · Structural optimization

C. Ch. Mitropoulou · N. D. Lagaros · M. Papadrakakis (✉)
Institute of Structural Analysis & Seismic Research, National Technical University Athens, Zografou Campus, 15780 Athens, Greece
e-mail: mpapadra@central.ntua.gr

C. Ch. Mitropoulou
e-mail: chmitrop@central.ntua.gr

N. D. Lagaros
e-mail: nlagaros@central.ntua.gr

1 Introduction

Most of the existing seismic design codes follow the prescriptive (or limit-state) concept, where the structure is considered safe and no collapse will occur if a number of checks, expressed mainly in terms of forces, are satisfied. A typical limit state based design can be viewed as one (ultimate strength) or two (serviceability and ultimate strength) limit state approach. According to a prescriptive design code, the strength of the structure is assessed in one limit state while a serviceability limit state is usually checked in order to ensure that the structure will not deflect or vibrate excessively during its functioning. The structures are allowed to absorb energy through inelastic deformation by designing them with reduced loading which is specified by the behaviour factor "q" also known as reduction factor "R". It is generally accepted that the capacity of a structure to resist seismic actions in the nonlinear range through energy dissipation permits their design for smaller seismic loads than those required for linear elastic response. The numerical verification of the behaviour factor was the subject of extensive research during the past decade (Fajfar 1998; Mazzolani and Piluso 1996; ATC-19 1995) in order to check for the validity of the design methodology and to make reliable predictions of the structural performance under extreme loading conditions.

A number of studies have been performed in the past dealing with structural optimization of reinforced concrete (RC) structures. One of the earliest studies on this subject is the work by Frangopol (1986) where the general formulation of the deterministic optimization problem was reviewed and a reliability-based optimization approach for the design of both steel and RC framed structures was presented. Moharrami and Grierson (1993) presented a computer-based method for the optimal design of RC buildings, where the width, depth and longitudinal reinforcement of member sections are considered as design variables. A review on the design of concrete structures can be found in the work by Sarma and Adeli (1998), where it was concluded that there is a need to perform further research on cost optimization of realistic three-dimensional structures with hundreds of members, where optimization can result in substantial savings. In the work by Li and Cheng (2001) the optimal decision model of the target value of performance-based structural system reliability of RC frames is established according to the cost-effectiveness criterion. Chan and Zou (2004) presented an optimization technique for the elastic and inelastic drift performance-based design of reinforced concrete buildings, while (Lagaros and Papadrakakis 2007) critically assessed the designs of a 3D reinforced concrete building, obtained according to the European seismic design code and a performance-based design procedure, in the framework of a multi-objective optimization problem. The results revealed that the designs based on the European seismic design code violated safety requirements for different hazard levels.

The main objective of this study is to examine the validity of the behaviour factor q in designing safe and economic RC structures using Eurocodes 2 and 8 (PrEN 2002; EN 2003). Numerical tests are performed on two different types of RC structures, a mid-rise irregular one and high-rise regular. The evaluation is performed on the basis of the initial and limit state costs designed to meet the EC2 and EC8 provisions. The designs for different values of q are compared with respect to the total cost resulting from the sum of the initial and the limit state cost. Limit state dependent cost, as considered in this study, represents monetary-equivalent losses in present values due to seismic events that are expected to occur during the design life of a new structure or the remaining life of an existing or a retrofitted structure. The limit state dependent cost consists of the damage cost, loss of contents, rental loss and income loss. The cost of the human fatality, that is associated with the limit-state dependent cost, is also accounted for in the present study.

2 Seismic design procedures

The current seismic design philosophy for RC structures relies on energy dissipation through inelastic deformations. Proper design of an earthquake resistant RC building should provide the structure with adequate deformation capacity to dissipate energy without a substantial reduction of its overall resistance against horizontal and vertical loading. According to EC2 and EC8, the fundamental design requirements that should be satisfied with an adequate degree of reliability are the requirements of no-collapse, damage limitation and minimum level of serviceability. In order to ensure that the structure will meet these requirements a number of checks must be satisfied: biaxial bending, shear forces, second-order effects (P-Δ effects), capacity design, limitation of inter-storey drift, stress levels, crack and deflection control. The study performed in this work is based on EC8 EN (2003) and EC2 PrEN (2002), with the following features: (1) The seismic load is an elastic response spectrum with 10% probability of being exceeded in 50 years (475 years return period), reduced by a behaviour factor q. (2) The nominal material strength is reduced by a factor $\gamma_s = 1.15$ for steel reinforcement and by a factor $\gamma_c = 1.50$ for concrete. (3) The analysis procedure employed is either the simplified modal or the multi-modal response spectrum analysis.

All EC2 checks must be satisfied for the gravity loads using the following load combination

$$S_d = 1.35 \sum_j G_{kj}{}'' + '' 1.50 \sum_i Q_{ki} \tag{1}$$

where "+" implies "to be combined with", the summation symbol "Σ" implies "the combined effect of", G_{kj} denotes the characteristic value "k" of the permanent action j and Q_{ki} refers to the characteristic value "k" of the variable action i. If the above constraints are satisfied, a multi-modal response spectrum analysis is performed and the earthquake loading is considered using the following load combination

$$S_d = \sum_j G_{kj}{}'' + '' E_d{}'' + '' \sum_i \psi_{2i} Q_{ki} \tag{2}$$

where E_d is the design value of the seismic action for the two components (longitudinal and transverse, respectively) computed from the design response spectrum and ψ_{2i} is the combination coefficient for the quasi-permanent action i, here taken equal to 0.30.

3 Response modification factors (*Q* and *R*)

Although, life safety under high seismic risk is the main objective of contemporary seismic design codes like EC8 EN (2003) and ATC-34 (1995), economic considerations permit the assumption that the structure will behave inelastically and could tolerate damages up to a certain level given that life safety is ensured. Since damage levels that a structure should tolerate cannot be predicted through linear analysis procedures, behavior or response modification factors are used in order to account for nonlinear response of structures. The behavior factors are used to scale down the linear elastic design response spectrum ordinates, corresponding to the maximum earthquake expected at the site, to the inelastic design response spectrum Palazzo and Petti (1996). Difficulties in evaluating behavior factors, that are generally applicable to various structural systems, materials, configurations and input motions, are well documented and the inherent drawbacks in code specified factors are widely accepted (Borzi and Elnashai 2000; Whittaker et al. 1999). No matter how difficult or unreliable may be the prediction of its value, the behavior factor is used both in European design codes denoted with q and in US design codes, named as response modification factor, denoted with R. The q factor is eventually an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic with 5% viscous damping, to the seismic design forces and is given by the following expression

$$q = q_0 \cdot k_D \cdot k_R \cdot k_W \geq 1.5 \tag{3}$$

where q_0 is the basic value of the behavior factor influenced by the structural configuration, the ductility class (k_D), the irregularity along the structural height (k_R), the main failure mode (k_W). According to the US codes the response modification factor consists of three parameters and is given by the following equation (Borzi and Elnashai 2000)

$$R = R_S \cdot R_\mu \cdot R_R \tag{4}$$

where R_S is the overstrength factor, R_μ is the ductility reduction factor and R_R is known as the redundancy factor, introduced to account for the number and distribution of active plastic hinges. The values of R are larger than those of q due to the differences on the applied ground motions (Borzi and Elnashai 2000). The theoretical background of reduction factors suffers from shortcomings due to the fact the physical mechanisms involved are not rigorously defined Kappos (1999). For these reasons many researchers have tried to introduce new definitions for q and R factors, trying to take into consideration the uncertainties involved (Lu et al. 2001; Miranda and Bertero 1994; Lam et al. 1998).

Using the reduction factor, a structural system is designed to have lower strength, in order to absorb energy through its inelastic deformations. Furthermore, since the demand in terms of inelastic deformations is expressed with reference to the ductility, there is a strict correlation between the reduction factor and the available ductility of the structure. Collapse mechanisms due to low energy dissipation should be avoided and adequate supply of local ductility should be provided in the plastic hinges. For this reason, three ductility levels (Low-Medium-High) are defined corresponding each to different structural requirements in order to ensure the desired ductility level. From the definition of the reduction factor, it is obvious that the higher the ductility level is the lower the seismic design loads are, corresponding to higher value of the reduction factor. Structures designed according to the low ductility class are imposed to higher seismic design actions leading to increased structural cost. However, due to their small ductility and their low inelastic response, less damage is expected. According to the Eurocodes (EC2 and EC8) a medium ductility level is recommended since it is considered as a compromise between resistance and economical design (Cuesta et al. 2003).

A number of methods have been proposed in the literature for evaluating the reduction factors based on two structural ingredients: (1) overstrength and (2) ductility. Miranda and Bertero (1994) have found that strength reduction factors are primarily influenced by the maximum tolerable displacement ductility demand, the period of the structural system and the soil conditions of the site. Lam et al. (1998) developed a relationship between the ductility reduction factor (R_μ) and the ductility for linear elasto-perfectly plastic SDOF systems (where $R = R_\mu$) in order to rationalize seismic design provisions for codes of practices. Borzi and Elnashai (2000) employed an earthquake set to derive values for force reduction factors needed for the structure to reach, and not exceed, a pre-determined level of ductility. It was observed that the force modification factors are only slightly influenced by the shape of the hysteretic model used in their derivation and that they are even less sensitive to strong motion characteristics. Cuesta et al. (2003) unified the results taken from two different approaches for determining the $R-\mu-\mathrm{T}$ relationships, where the ground motion frequency content is considered. Recently, Lee and Foutch (2006) used different R factors to design steel moment resisting frame structures. In their study it was found that the recommended R factors provide conservative designs for some of the structures considered. Karavasilis et al. (2007) proposed simplified expressions to estimate the behaviour factor of plane steel moment resisting frames, based on statistical analysis of the results of nonlinear dynamic analyses. Karakostas et al. (2007) derived the ductility component of the behavior factor from statistical analysis of constant ductility spectra, and proposed empirical relationships suitable for design purposes. All these studies however, do not reach any concrete conclusion regarding the reliability of the design philosophy based on the behavior factors.

4 Problem definition

A number of studies have been published on the optimum design of RC structures (Frangopol 1986; Moharrami and Grierson 1993; Sarma and Adeli 1998; Li and Cheng 2001; Chan and Zou 2004; Lagaros and Papadrakakis 2007) taking into account different types of constraints during the design optimization procedure. However, none of them has taken into consideration the behaviour factor q. The main scope of this study is to examine the influence of the behaviour factor q in designing safe and economic structures. For this reason optimally designed structures for different values of q are compared with reference to the total life cycle cost and their performance is critically assessed.

4.1 Formulation of the optimization problem

The mathematical formulation of the optimization problem considered in the present work is defined as follows:

$$\begin{array}{ll} \min_{\mathbf{s}\in\mathsf{F}} & C_{IN}(\mathbf{s}) \\ \text{where} & C_{IN}(\mathbf{s}) = C_b(\mathbf{s}) + C_{sl}(\mathbf{s}) + C_{cl}(\mathbf{s}) + C_{ns}(\mathbf{s}) \\ \text{subject to} & g_j^{SERV}(\mathbf{s}) \leq 0 \quad j = 1, \ldots, m \\ & g_j^{ULT}(\mathbf{s}) \leq 0 \quad j = 1, \ldots, k \end{array} \tag{5}$$

where $\mathbf{s}$ represents the design vector corresponding to the dimensions of columns and beams cross-sections, F is the feasible region where all the serviceability and ultimate constraint functions (g^{SERV} and g^{ULT}) are satisfied. In this work the boundaries of the feasible region are defined according to the recommendations of the EC8. The objective function considered is the total initial construction cost of the structure C_{IN}, while $C_b(\mathbf{s})$, $C_{sl}(\mathbf{s})$, $C_{cl}(\mathbf{s})$ and $C_{ns}(\mathbf{s})$ correspond to the total initial construction cost of beams, slabs, columns and non structural elements, respectively. The term "initial cost" of a new structure corresponds to the cost just after construction. The initial cost is related to material, which includes concrete, steel reinforcement, and labour costs for the construction of the building. The solution of the resulting optimization problem is performed by means of Evolutionary Algorithms (EA). The optimization procedure was implemented for four characteristic values of behaviour factor q in two different buildings resulting into eight optimum designs. In all eight cases the design procedure was based on linear elastic static analysis according to EC8. The designs obtained through the optimization procedure are assessed by performing life-cycle cost analysis.

4.2 Solving the optimization problem

Evolutionary Algorithms (EA) are population based, probabilistic, direct search optimization algorithms gleaned from principles of Darwinian evolution. Starting with an initial population of μ candidate designs, an offspring population of λ designs is created from the parents using variation operators. Depending on the manner in which the variation and selection operators are designed and the spaces in which they act, different classes of EA have been proposed. In the EA algorithm employed in this study Lagaros et al. (2008), each member of the population is equipped with a set of parameters:

$$\begin{aligned} \mathbf{a} &= [(\mathbf{s}_\mathrm{d}, \boldsymbol{\gamma}), (\mathbf{s}_\mathrm{c}, \boldsymbol{\sigma}, \boldsymbol{\alpha})] \in (\mathrm{I_d I_c}) \\ \mathrm{I_d} &= D_\mathrm{d}^\mathrm{n} \times R_+^{\mathrm{n}_\gamma} \\ \mathrm{I_c} &= R_\mathrm{c}^\mathrm{n} \times R_+^{\mathrm{n}_\sigma} \times [-\pi, \pi]^{\mathrm{n_a}} \end{aligned} \tag{6}$$

where $\mathbf{s}_\mathrm{d}$ and $\mathbf{s}_\mathrm{c}$ are the vectors of discrete and continuous design variables defined in the discrete and continuous design sets D_d^n and R^{n_c}, respectively. Vectors $\boldsymbol{\gamma}$, $\boldsymbol{\sigma}$ and $\boldsymbol{\alpha}$ are the distribution parameter vectors taking values in $R_+^{\mathrm{n}_\gamma}$, $R_+^{\mathrm{n}_\sigma}$ and $[-\pi, \pi]_a^\mathrm{n}$, respectively. Vector $\boldsymbol{\gamma}$ corresponds to the variances of the Poisson distribution. Vector $\boldsymbol{\sigma} \in R_+^{n_\sigma}$ corresponds to the standard deviations ($1 \leq \mathrm{n}_\sigma \leq \mathrm{n_c}$) of the normal distribution. Vector $\boldsymbol{\alpha} \in [-\pi, \pi]_a^\mathrm{n}$ is related to the inclination angles ($\mathrm{n}_\alpha = (\mathrm{n_c} - \mathrm{n}_\sigma/2)(\mathrm{n}_\sigma - 1)$) defining linearly correlated mutations of the continuous design variables $\mathbf{s}_\mathrm{c}$, where $n = n_d + n_c$ is the total number of design variables.

Let $\mathrm{P}(t) = \{\mathrm{a}_1, \ldots, \mathrm{a}_\mu\}$ denotes a population of individuals at the tth generation. The genetic operators used in the EA method are denoted by the following mappings:

$$\begin{aligned} &\mathrm{rec}: \quad (\mathrm{I_d}, \mathrm{I_c})^\mu \rightarrow (\mathrm{I_d}, \mathrm{I_c})^\lambda (\text{recombination}) \\ &\mathrm{mut}: \quad (\mathrm{I_d}, \mathrm{I_c})^\lambda \rightarrow (\mathrm{I_d}, \mathrm{I_c})^\lambda (\text{mutation}) \\ &\mathrm{sel}_\mu^\mathrm{k}: \quad (\mathrm{I_d}, \mathrm{I_c})^\mathrm{k} \rightarrow (\mathrm{I_d}, \mathrm{I_c})^\mu (\text{selection}, \mathrm{k} \in \{\lambda, \mu + \lambda\}) \end{aligned} \tag{7}$$

A single iteration of the EA, which is a step from the population $P_p^{(t)}$ to the next parent population $P_p^{(t+1)}$, is modelled by the mapping

$$\mathrm{opt_{EA}}: \quad (\mathrm{I_d}, \mathrm{I_c})_\mathrm{t}^\mu \rightarrow (\mathrm{I_d}, \mathrm{I_c})_{t+1}^\mu \tag{8}$$

5 Life-cycle cost analysis

The total cost C_{TOT} of a structure, may refer either to the design-life period of a new structure or to the remaining life period of an existing or retrofitted structure. This cost can be expressed as a function of time and the design vector as follows (Wen and Kang 2001)

$$C_{TOT}(t, s) = C_{IN}(s) + C_{LS}(t, s) \tag{9}$$

where C_{IN} is the initial cost of a new or retrofitted structure, C_{LS} is the present value of the limit state cost; s is the design vector corresponding to the design loads, resistance and material properties, while t is the time period. The term "initial cost" of a new structure refers to the cost just after construction. The initial cost is related to the material and the labour cost for the construction of the building which includes concrete, steel reinforcement, labour cost for placement as well as the non-structural component cost. The term "limit state cost" refers to the potential damage cost from earthquakes that may occur during the life of the structure.

Table 1 Limit state drift ratio limits for bare moment resisting frames and contents DI

Limit state	Interstorey drift (%) Ghobarah (2004)	Contents DI (g) Elenas and Meskouris (2001)
(I) - None	$\theta \leq 0.1$	$\ddot{u}_{\text{floor}} \leq 0.05$
(II) - Slight	$0.1 < \theta \leq 0.2$	$0.05 < \ddot{u}_{\text{floor}} \leq 0.10$
(III) - Light	$0.2 < \theta \leq 0.4$	$0.10 < \ddot{u}_{\text{floor}} \leq 0.20$
(IV) - Moderate	$0.4 < \theta \leq 1.0$	$0.20 < \ddot{u}_{\text{floor}} \leq 0.80$
(V) - Heavy	$1.0 < \theta \leq 1.8$	$0.80 < \ddot{u}_{\text{floor}} \leq 0.98$
(VI) - Major	$1.8 < \theta \leq 3.0$	$0.98 < \ddot{u}_{\text{floor}} \leq 1.25$
(VII) - Collapsed	$\theta > 3.0$	$\ddot{u}_{\text{floor}} > 1.25$

It accounts for the cost of the repairs after an earthquake, the cost of loss of contents, the cost of injury recovery or human fatality and other direct or indirect economic losses related to loss of contents, rental and income. The quantification of the losses in economical terms depends on several socio-economic parameters. It should be mentioned that in the calculation formula of C_{LS} a regularization factor is used that transforms the costs in present values. The most difficult cost to quantify is the cost corresponding to the loss of a human life. In this work the fatality cost is based on the legal damages that the courts adjudge in the case of loss of life.

Damage may be quantified by using several damage indices (DIs) whose values can be related to particular structural damage states. The idea of describing the state of damage of the structure by a specific number, on a defined scale in the form of a damage index, is attractive because of its simplicity. So far a significant number of researchers has studied various damage indices for reinforced concrete or steel structures, a detailed survey can be found in the work by Ghobarah et al. (1999). Damage, in the context of life cycle cost analysis (LCCA), means not only structural damage but also non-structural damage. The latter including the case of architectural damage, mechanical, electrical and plumbing damage and also the damage of furniture, equipment and other contents. The maximum interstory drift (θ) has been considered as the response parameter which best characterises the structural damage, which has been associated with all types of losses. It is generally accepted that interstorey drift can be used as one limit state criterion to determine the expected damage. The relation between the drift ratio limits with the limit state, employed in this study (Table 1), is based on the work of Ghobarah (2004) for ductile RC moment resisting frames. On the other hand, the intensity measure associated with the loss of contents like furniture and equipment is the maximum response floor acceleration. The relation of the limit state with the values of the floor acceleration used in this work (Table 1) are based on the work of Elenas and Meskouris (2001).

The limit state cost (C_{LS}), for the i-th limit state, can thus be expressed as follows

$$C_{LS}^{i,\theta} = C_{dam}^{i} + C_{con}^{i,\theta} + C_{ren}^{i} + C_{inc}^{i} + C_{inj}^{i} + C_{fat}^{i} \tag{10a}$$

$$C_{LS}^{i,acc} = C_{con}^{i,acc} \tag{10b}$$

where C_{dam}^{i} is the damage repair cost, $C_{con}^{i,\theta}$ is the loss of contents cost due to structural damage that is quantified by the maximum interstorey drift, C_{ren}^{i} is the loss of rental cost, C_{inc}^{i} is the income loss cost, C_{inj}^{i} is the cost of injuries and C_{fat}^{i} is the cost of human fatality. These cost components are related to the damage of the structural system. $C_{con}^{i,acc}$ is the loss

Table 2 Limit state costs-calculation formula (Ellingwood and Wen 2005; ATC-13 1985; FEMA 227 1992; Kang and Wen 2000)

Cost category	Calculation formula	Basic cost
Damage/repair (C_{dam})	Replacement cost × floor area × mean damage index	$1,500$ €/m^2
Loss of contents (C_{con})	Unit contents cost × floor area × mean damage index	500 €/m^2
Rental (C_{ren})	Rental rate × gross leasable area × loss of function	10 €/month/m^2
Income (C_{inc})	Rental rate × gross leasable area × down time	$2,000$ €/year/m^2
Minor Injury ($C_{inj,m}$)	Minor injury cost per person × floor area × occupancy rate* × expected minor injury rate	2,000 €/person
Serious Injury ($C_{inj,s}$)	Serious injury cost per person × floor area × occupancy rate* × expected serious injury rate	2×10^4 €/person
Human fatality (C_{fat})	Human fatality cost per person × floor area × occupancy rate* × expected death rate	2.8×10^6 €/person

* Occupancy rate 2 persons/100 m^2

Table 3 Limit state parameters for cost evaluation

Limit state	FEMA 227 (1992)				ATC-13 (1985)	
	Mean damage index (%)	Expected minor injury rate	Expected serious injury rate	Expected death rate	Loss of function (%)	Down time (%)
(I) - None	0	0	0	0	0	0
(II) - Slight	0.5	3.0E-05	4.0E-06	1.0E-06	0.9	0.9
(III) - Light	5	3.0E-04	4.0E-05	1.0E-05	3.33	3.33
(IV) - Moderate	20	3.0E-03	4.0E-04	1.0E-04	12.4	12.4
(V) - Heavy	45	3.0E-02	4.0E-03	1.0E-03	34.8	34.8
(VI) - Major	80	3.0E-01	4.0E-02	1.0E-02	65.4	65.4
(VII) - Collapsed	100	4.0E-01	4.0E-01	2.0E-01	100	100

of contents cost due to floor acceleration (Elenas and Meskouris 2001). Details about the calculation formula for each limit state cost along with the values of the basic cost for each category can be found in Table 2 (Wen and Kang 2001; Lagaros 2007). The values of the mean damage index, loss of function, down time, expected minor injury rate, expected serious injury rate and expected death rate used in this study are based on (Ellingwood and Wen 2005; ATC-13 1985; FEMA 227 1992; Kang and Wen 2000). Table 3 provides the (ATC-13 1985; FEMA 227 1992) limit state dependent damage consequence severities.

Based on a Poisson process model of earthquake occurrences and an assumption that damaged buildings are immediately retrofitted to their original intact conditions after each major damage-inducing seismic attack, Wen and Kang (2001) proposed the following formula for the limit state cost function considering N limit states

$$C_{LS} = C_{LS}^{\theta} + C_{LS}^{acc} \tag{11a}$$

$$C_{LS}^{\theta}(t, \mathbf{s}) = \frac{\nu}{\lambda}\left(1 - e^{-\lambda t}\right)\sum_{i=1}^{N} C_{LS}^{i,\theta} \cdot P_i^{\theta} \tag{11b}$$

$$C_{LS}^{acc}(t, \mathbf{s}) = \frac{\nu}{\lambda}\left(1 - e^{-\lambda t}\right)\sum_{i=1}^{N} C_{LS}^{i,acc} \cdot P_i^{acc} \tag{11c}$$

where

$$P_i^{DI} = P(DI > DI_i) - P(DI > DI_{i+1}) \tag{12}$$

and

$$P(DI > DI_i) = (-1/t) \cdot \ln[1 - \bar{P}_i(DI - DI_i)] \tag{13}$$

P_{i} is the probability of the ith limit state being violated given the earthquake occurrence and C_{LS}^{i} is the corresponding limit state cost; $P(DI - DI_i)$ is the exceedance probability given occurrence; DI_i, DI_{i+1} are the damage indices (maximum interstorey drift or maximum floor acceleration) defining the lower and upper bounds of the ith limit state; $\bar{P}_i(DI - DI_i)$ is the annual exceedance probability of the maximum damage index DI_i; ν is the annual occurrence rate of significant earthquakes modelled by a Poisson process and t is the service life of a new structure or the remaining life of a retrofitted structure. Thus, for the calculation of the limit state cost of Eq. 11b the maximum interstorey drift DI is considered, while for the case of Eq. 11b the maximum floor acceleration is used. The first component of Eqs. 11b or 11c, with the exponential term, is used in order to express C_{LS} in present value, where λ is the annual monetary discount rate. In this work the annual monetary discount rate λ is taken to be constant, since considering a continuous discount rate is accurate enough for all practical purposes according to (Rackwitz 2006; Rackwitz et al. 2005). Various approaches yield values of the discount rate λ in the range of 3–6% (Ellingwood and Wen 2005), in this study it was taken equal to 5%.

Each limit state is defined by drift ratio limits or floor acceleration, as listed in Table 1. When one of the *DIs* is exceeded the corresponding limit state is assumed to be reached. The annual exceedance probability $\bar{P}_i(DI > DI_i)$ is obtained from a relationship of the form

$$\bar{P}_i(DI > DI_i) = \gamma(DI_i)^{-k} \tag{14}$$

The above expression is obtained by best fit of known $\bar{P}_i - DI_i$ pairs for each of the two *DIs*. These pairs correspond to the 2, 10 and 50% in 50 years earthquakes that have known probabilities of exceedance $\bar{P}_i$. In this work the maximum value of DI_i (interstorey drift or floor acceleration) corresponding to the three hazard levels considered, are obtained through a number of non-linear dynamic analyses. The selection of the proper external loading for design and/or assessment purposes is not an easy task due to the uncertainties involved in the seismic loading. For this reason a rigorous treatment of the seismic loading is to assume that the structure is subjected to a set of records that are more likely to occur in the region where the structure is located. In our case as a series of twenty artificial accelerograms per hazard level is implemented.

According to Poisson's law the annual probability of exceedance of an earthquake with a probability of exceedance p in t years is given by the formula

$$\bar{P} = (-1/t) \cdot \ln(1 - p) \tag{15}$$

This means that the 2/50 earthquake has a probability of exceedance equal to $\bar{P}_{2\%} = -\ln(1-0.02)/50 = 4.04 \times 10^{-4}(4.04 \times 10^{-2}\%)$.

6 Non-linear dynamic analysis procedure

In order to define the $\bar{P}_i - DI_i$ pairs for the two *DIs,* three performance levels are considered and twenty ground motion records are selected for each hazard level. Sixty non-linear dynamic analyses have to be performed for each candidate design in order to assess its performance in the framework of LCCA. Although, strong motion recording programs have been carried out in US over the last 60 years, and more recently in many parts of the world; for many applications, it is necessary to estimate future seismic records in multiple hazard levels at a particular site, which is often outside the existing recorded data. For the present investigation a number of records for each 50/50, 10/50 and 2/50 hazard level were required for the region of Eastern Mediterranean. These records were not possible to be found in any database, thus artificial ground motions, compatible with the corresponding design spectrum, were generated for each hazard level. Apart from enriching the catalogues of earthquake records of a region, a second reason for using artificial records is that they usually present a larger number of cycles than natural records, and therefore they result in more severe earthquake loading for a greater range of structural periods. Although one may argue that artificial records are unrealistic and thus they do not simulate an earthquake, they can still be considered as an envelope ground motion and hence as a more conservative loading scenario. Seven uncorrelated artificial accelerograms for each hazard level, produced from the smooth EC8 elastic spectra (Fig. 1a), are used in the present study. The peak ground acceleration for the three hazard levels considered are obtained from the log-linear equation of (Papazachos et al. 1993)

$$PGA = e^{a+b\times\ln(T_m)}(/1000)[\mathrm{m/s^2}] \tag{16}$$

where seismic hazard parameters a and b are equal to 4.01 and 0.61 respectively, while T_m is equal to 72, 475 and 2,475 years for the 50/50, 10/50 and 2/50 hazard levels respectively. All twenty artificial accelerograms are consistent with the corresponding elastic response spectrum of Eurocode 8 EN (2003) and have been simulated using the SIMQKE computer program of Gasparini and Vanmarcke (1976). The design spectrum used for the generation of the artificial records correspond to soil type B (characteristic periods $\mathrm{T_B} = 0.15$ s, $\mathrm{T_C} = 0.50$ s and $\mathrm{T_D} = 2.00$ s). Moreover, the importance factor γ_I was taken equal to 1.0, while the damping correction factor η is equal to 1.0, since a damping ratio of 5% has been considered. For the generation of all artificial accelerograms the trapezoidal time envelope was used. The PGA values along with the duration and the time steps Δt of the artificial accelerograms for the 50/50, 10/50 and 2/50 hazard levels are given in Table 4. Three typical artificial accelerograms corresponding to the three hazard levels are shown in Fig. 1b. In order to account for the randomness of the incident angle, the artificial accelerograms are applied with a random angle with respect to the structural axes. The incident angles for the seven artificial accelerograms are generated using the Latin Hypercube Sampling method Olsson et al. (2003) in the range of 0–180°.

7 Test examples

In this work, two 3D RC moment resisting framed (MRF) buildings have been considered in order to study the influence of the behaviour factor q on the design of RC buildings. The first

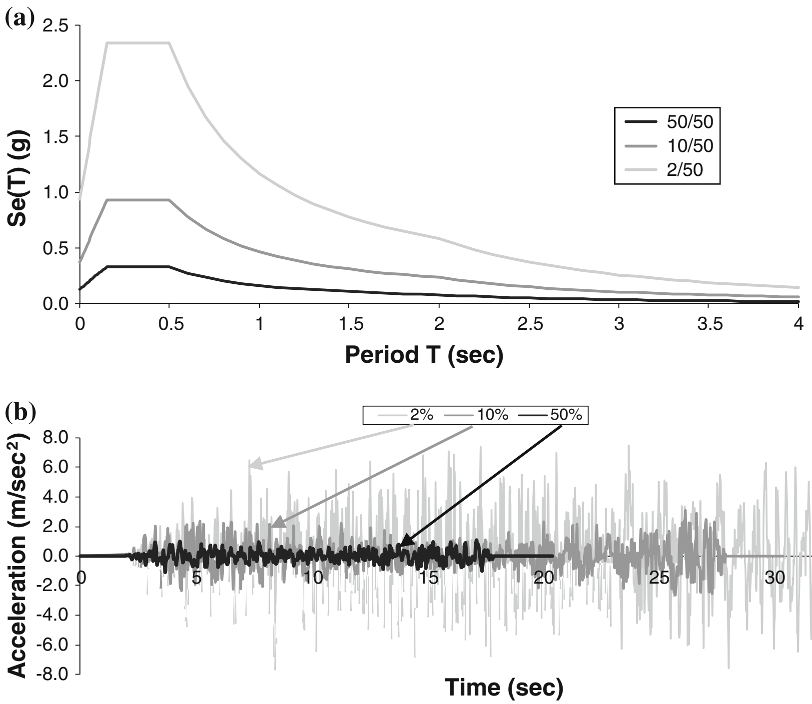

Fig. 1 **a** EC8 elastic 5%-damped spectra of the three hazard levels and **b** three typical artificial accelerograms

Table 4 Artificial ground motion characteristics

Hazard level	PGA (g)	Duration (s)	No of steps	Δt (s)
50/50	0.11	20.5	1024	2.00E-02
10/50	0.31	30.5	2048	1.49E-02
2/50	0.78	35.5	4096	8.67E-03

test example is a five storey RC building with non-symmetrical plan view while the second one is an eight storey RC building having symmetrical plan view. Both buildings have been designed to meet the Eurocode requirements, i.e. the EC8 (EN 2003) and EC2 (PrEN 2002) design codes. Concrete of class C20/25 (nominal cylindrical strength of 20 MPa) and class S500 steel (nominal yield stress of 500 MPa) are assumed. The base shear is obtained from the response spectrum for soil type B (characteristic periods $T_B = 0.15$ s, $T_C = 0.50$ s and $T_D = 2.00$ s) while the PGA considered is equal to 0.31 g. Moreover, the importance factor γ_I was taken equal to 1.0, while the damping correction factor is equal to 1.0, since a damping ratio of 5% has been considered.

The slab thickness is equal to 15 cm, for both test examples, while it is considered to contribute to the moment of inertia of the beams with an effective flange width. In addition to the self weight of beams and slabs, a distributed permanent load of $2\,\text{kN/m}^2$ due to floor finishing-partitions and an imposed load with nominal value of $1.5\,\text{kN/m}^2$, are considered. The nominal permanent and imposed loads are multiplied by load factors of 1.35 and 1.5,

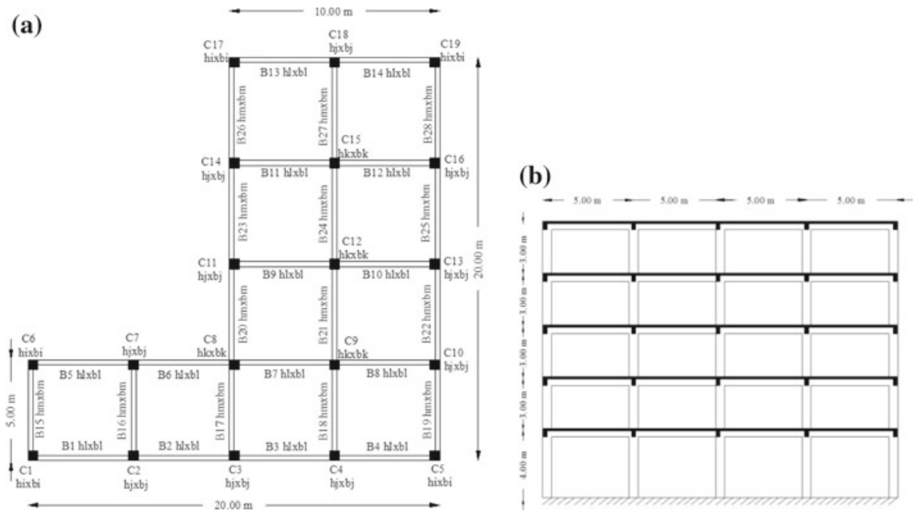

Fig. 2 Five storey test example—**a** plan view, **b** front view

respectively. Following EC8, in the seismic design combination, dead loads are considered with their nominal values, while live loads with the 30% of their nominal value.

In both test examples the parametric study is performed in two stages: (1) design based on structural optimization and (2) life-cycle cost analysis. In the first stage the optimum design is computed employing the EA($\mu + \lambda$) optimization scheme (Lagaros et al. 2008) with ten parent and offspring ($\mu = \lambda = 10$) design vectors for both test examples. Four optimization problems are defined, following the design recommendations of EC8 and EC2, according to the value of the behaviour factor considered. The optimum designs obtained are labelled as $D_{q=a}$ defining the value of the behaviour factor used. In all formulations the initial construction cost is the objective function to be minimized. The columns and beams are of rectangular cross-sectional shape, and are separated into groups. The two dimensions of the columns/beams along with the longitudinal, transverse reinforcement and its spacing are the five design variables that are assigned to each group of the columns/beams. In the second stage, life-cycle cost analysis is performed on the optimum designs by means of non-linear dynamic analysis where the beam-column members are modelled with the inelastic force-based fibre element (Papaioannou et al. 2005).

7.1 Five storey non-symmetrical test example

The plan and front views of the five storey non-symmetrical test example are shown in Fig. 2. The structural elements (beams and columns) are separated into 10 groups, 8 for the columns and 2 for the beams, resulting into 50 design variables. The optimum designs achieved for different values of the q factor are presented in Table 5. It can be seen that the initial construction cost of design $D_{q=1}$ is increased by the marginal quantity of 7% compared to $D_{q=2}$, while it is 10 and 12% more expensive compared to $D_{q=3}$ and $D_{q=4}$, respectively. It can therefore be said that the initial cost of RC structures, designed on the basis of their elastic response for the design earthquake, is not excessive taking into consideration the additional costs of a building structure which are practically the same for all designs $q = 1$ to 4. When the four designs are compared with respect to the cost of the RC skeletal members, design

Table 5 Five storey test example-optimum designs obtained for different values of behaviour factor *q*

		Optimum designs			
		q = 1	q = 2	q = 3	q = 4
Columns	h1×b1	0.80×0.80, LR: 34Ø32, TR: (4)Ø10/10 cm	0.60×0.60, LR:8Ø24+ 12Ø28, TR: (4)Ø10/20 cm	0.55×0.55, LR:8Ø20+ 12Ø24, TR: (2)Ø10/20 cm	0.55×0.55, LR:8Ø24+ 4Ø28, TR: (2)Ø10/20 cm
	h2×b2	0.85×0.85, LR: 34Ø32, TR: (4)Ø10/10 cm	0.60×0.60, LR:8Ø24+ 12Ø28, TR: (4)Ø10/20 cm	0.55×0.55, LR:8Ø22+ 12Ø26, TR: (2)Ø10/20 cm	0.55×0.55, LR:8Ø24+ 4Ø28, TR: (2)Ø10/20 cm
	h3×b3	0.80×0.80, LR: 28Ø32, TR: (4)Ø10/10 cm	0.60×0.60, LR:8Ø24+ 12Ø28, TR: (4)Ø10/20 cm	0.50×0.50, LR:4Ø22+ 12Ø26, TR: (2)Ø10/20 cm	0.50×0.50, LR:4Ø26+ 4Ø32, TR: (2)Ø10/20cm
	h4×b4	0.70×0.70, LR:8Ø22+ 12Ø26, TR: (4)Ø10/10 cm	0.55×0.55, LR:8Ø24+ 12Ø28, TR: (4)Ø10/20 cm	0.55×0.55, LR:8Ø18+ 4Ø22, TR: (2)Ø10/20 cm	0.55×0.55, LR:8Ø18+ 4Ø22, TR: (2)Ø10/20 cm
	h5×b5	0.70×0.70,LR: 26Ø32, TR: (4)Ø10/10 cm	0.55×0.55, LR:4Ø28+ 8Ø24, TR: (4)Ø10/20 cm	0.55×0.55, LR:8Ø24+ 4Ø28, TR: (2)Ø10/20 cm	0.55×0.55, LR:8Ø20+ 4Ø24, TR: (2)Ø10/20 cm
	h6×b6	0.70×0.70, LR: 24Ø32, TR: (4)Ø10/10 cm	0.50×0.55, LR:12Ø28+ 8Ø24, TR: (4)Ø10/20 cm	0.45×0.45, LR:4Ø24+ 4Ø28, TR: (2)Ø10/20 cm	0.45×0.45, LR:4Ø26+ 4Ø32, TR: (2)Ø10/20 cm
	h7×b7	0.65×0.65, LR:15Ø18+ 16Ø20, TR: (4)Ø10/10 cm	0.35×0.60, LR:8Ø18+ 8Ø20, TR: (2)Ø10/20 cm	0.35×0.55, LR:7Ø16+ 5Ø20, TR: (2)Ø10/20 cm	0.50×0.30, LR:5Ø18+ 6Ø16, TR: (2)Ø10/20 cm
	h8×b8	0.60×0.65, LR:24Ø20+ 20Ø18, TR:(4)Ø10/10 cm	0.40×0.60, LR: 18Ø18, TR: (2)Ø10/20 cm	0.35×0.55, LR:8Ø18+ 5Ø20, TR: (2)Ø10/20 cm	0.55×0.30, LR:8Ø18, TR: (2)Ø10/20 cm
Beams	h9×b9	0.45×0.55,LR: 15Ø20, TR: (2)Ø10/10 cm	0.30×0.50, LR: 9Ø18, TR: (2)Ø10/20 cm	0.30×0.55, LR:3Ø20+ 4Ø14, TR: (2)Ø10/20 cm	0.25×0.45, LR:4Ø16+ 4Ø14, TR: (2)Ø10/20 cm
	h10×b10	0.50×0.55, LR: 24Ø18, TR: (2)Ø8/15 cm	0.30×0.55, LR: 10Ø18, TR: (2)Ø8/15 cm	0.30×0.55, LR:6Ø20, TR: (2)Ø8/15 cm	0.25×0.45, LR:4Ø16, TR: (2)Ø8/15 cm
$C_{IN,RC\ Frame}$ (1,000€)		1.85E+02	1.32E+02	1.11E+02	9.62E+01
C_{IN} (1,000€)		8.10E+02	7.57E+02	7.36E+02	7.21E+02

$D_{q=1}$ is increased by 40% compared to $D_{q=2}$ and by 67 and 92% compared to $D_{q=3}$ and $D_{q=4}$, respectively.

Table 3 provides the (ATC-13 1985; FEMA 227 1992) limit state dependent parameters required for the calculation of the following costs: damage repair, loss of contents, loss of rental, income loss, cost of injuries and that of human fatality. The computed components of the limit states, for the five storey RC building, are listed in Table 6. From Table 6 it can be seen that damage and income loss costs are the dominating cost components for the limit states I through VI representing, by average, the 83% of C^i_{LS}, while the cost of human fatality is the dominant cost (73%) at the highest limit state VII. Below it is explained the calculation procedure of the limit state cost taking as an example the design corresponding to $D_{q=3}$. In

Table 6 Five storey test example—limit state cost components (1,000 €)

Limit state	C^i_{dam}	C^i_{con}	C^i_{ren}	C^i_{inc}	C^i_{inj}		C^i_{fat}	$C^i_{LS}(I)$	$C^i_{LS}(II)$
					Minor	Serious			
I	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
II	9.38	3.13	1.35	22.50	0.00	0.00	0.07	36.35	36.42
III	93.75	31.25	5.00	83.25	0.02	0.02	0.70	213.25	213.98
IV	375.00	125.00	18.60	310.00	0.15	0.20	7.00	828.60	835.95
V	843.75	281.25	52.20	870.00	1.50	2.00	70.00	2047.20	2120.70
VI	1500.00	500.00	98.10	1635.00	15.00	20.00	700.00	3733.10	4468.10
VII	1875.00	625.00	150.00	2500.00	20.00	200.00	14000.00	5150.00	19370.00

the first step three $(\bar{P}_i - \theta_i)$ and three $(\bar{P}_i - \ddot{u}_{floor,i})$ pairs are defined corresponding to the three hazard levels

$$\begin{aligned}
\bar{P}_{50\%} &= 1.39\% \quad \theta_{50\%} = 0.14\,\% \quad \ddot{u}_{floor,50\%} = 0.36\,\mathrm{g} \\
\bar{P}_{10\%} &= 2.10 \times 10^{-1}\% \quad \theta_{10\%} = 0.42\,\% \quad \ddot{u}_{floor,10\%} = 0.96\,\mathrm{g} \\
\bar{P}_{2\%} &= 4.04 \times 10^{-2}\% \quad \theta_{2\%} = 1.24\,\% \quad \ddot{u}_{floor,2\%} = 2.18\,\mathrm{g}
\end{aligned} \tag{17}$$

The *abscissa* values for both $(\bar{P}_i - \theta_i)$ and $(\bar{P}_i - \ddot{u}_{floor,i})$ pairs, corresponding to the median values of the maximum interstorey drifts and maximum floor accelerations for the three hazard levels in question, are obtained through 20 non-linear time history analyses performed for each hazard level 50/50, 10/50 and 2/50. The median values of the four designs are shown in Fig. 3a, b. The *ordinate* values, corresponding to the annual probabilities of exceedance, are calculated using Eq. 15. Subsequently, exponential functions for the two *DIs*, as the one described in Eq. 14, is fitted to the pairs of Eq. 17. Once the two functions of the best fitted curve are defined the annual probabilities of exceedance $\bar{P}_i$ for each of the seven limit states of Table 1 are calculated. Substituting $\bar{P}_i$ into Eq. 13 the exceedance probabilities of the limit state given occurrence are computed and the probabilities P_i are then evaluated from Eq. 12. This procedure is performed for each one of the DIs, i.e. interstorey drifts and floor accelerations. The limit state cost of Eq. 11a is calculated adding the two components of Eqs. 11b and (11c), while these two components are defined combining the numerical values of the cost components of Table 6 and the corresponding probabilities P_i.

Figure 4 depicts the optimum designs obtained with reference to the behaviour factor, along with the initial construction, limit state and total life-cycle costs. It can be observed from this figure that although design $D_{q=1}$ is worst, compared to the other three designs with reference to C_{IN}, with respect to C_{TOT} the design $D_{q=4}$ is the most expensive. Comparing design $D_{q=3}$, obtained for the behaviour factor suggested by the Eurocodes for RC buildings, with reference to C_{TOT}, it can be seen that it is 50% and 20% more expensive compared to $D_{q=1}$ and $D_{q=2}$, respectively; while it is 10% less expensive compared to $D_{q=4}$.

The contribution of the initial and limit state cost components to the total life-cycle cost are shown in Fig. 5. C_{IN} represents the 75% of the total life-cycle cost for design $D_{q=1}$ while for designs $D_{q=2}$, $D_{q=3}$ and $D_{q=4}$ represents the 59, 50 and 45%, respectively. Although the initial cost is the dominant contributor for all optimum design; for design $D_{q=1}$ the second dominant contributor is the cost of contents due to floor acceleration while for designs $D_{q=2}$, $D_{q=3}$ and $D_{q=4}$ damage and income costs are almost equivalent representing the

Table 7 Eight storey test example-optimum designs obtained for different values of behaviour factor *q*

		Optimum designs			
		q = 1	q = 2	q = 3	q = 4
Columns	h1×b1	1.35×1.35, LR: 88Ø30, TR: (5)Ø10/10 cm	0.80×0.75, LR: 26Ø32, TR: (4)Ø10/15 cm	0.80×0.60, LR: 24Ø28, TR: (4)Ø10/20 cm	0.80×0.60, LR:10Ø24+ 12Ø28, TR: (2)Ø10/20 cm
	h2×b2	0.90×0.85, LR: 34Ø30, TR: (4)Ø10/10 cm	0.80×0.80, LR: 30Ø32, TR: (4)Ø10/10 cm	0.65×0.60, LR: 26Ø28, TR: (4)Ø10/20 cm	0.75×0.35, LR:8Ø22+ 12Ø26, TR: (2)Ø10/20 cm
	h3×b3	1.05×1.10, LR: 50Ø30, TR: (4)Ø10/10 cm	0.80×0.80, LR: 30Ø32, TR: (4)Ø10/10 cm	0.75×0.65, LR: 28Ø28, TR: (4)Ø10/20 cm	0.75×0.70, LR:10Ø22+ 12Ø26, TR: (2)Ø10/20 cm
	h4×b4	1.10×1.05, LR: 60Ø30, TR: (4)Ø10/10 cm	0.80×0.85, LR: 32Ø32, TR: (4)Ø10/10 cm	0.75×0.65, LR: 30Ø28, TR: (4)Ø10/15 cm	0.70×0.65, LR:8Ø26+ 12Ø32, TR: (2)Ø10/20 cm
	h5×b5	0.85×0.85, LR: 36Ø30, TR: (4)Ø10/10 cm	0.65×0.65, LR: 8Ø22+ 12Ø26, TR: (4)Ø10/10 cm	0.65×0.55,LR: 8Ø24+4Ø26, TR: (2)Ø10/20 cm	0.60×0.55, LR:8Ø20+ 4Ø24, TR: (2)Ø10/20 cm
	h6×b6	0.80×0.80, LR: 36Ø30, TR: (4)Ø10/10 cm	0.65×0.65, LR: 22Ø32, TR: (2)Ø10/10 cm	0.55×0.55, LR:8Ø24+ 12Ø28, TR: (2)Ø10/20 cm	0.55×0.40, LR:6Ø22+ 12Ø26, TR: (2)Ø10/20 cm
	h7×b7	0.85×0.85, LR: 36Ø30, TR: (4)Ø10/10 cm	0.65×0.65, LR: 24Ø32, TR: (2)Ø10/10 cm	0.75×0.50, LR: 24Ø28, TR: (2)Ø10/20 cm	0.75×0.55, LR:10Ø22+ 12Ø26, TR: (2)Ø10/20 cm
	h8×b8	0.85×0.85, LR: 40Ø30, TR: (4)Ø10/10 cm	0.70×0.70, LR: 24Ø32, TR: (4)Ø10/10 cm	0.55×0.60, LR: 22Ø28, TR: (2)Ø10/20 cm	0.55×0.55, LR:8Ø24+ 12Ø28, TR: (2)Ø10/20 cm
	h9×b9	0.60×0.60, LR: 8Ø26+ 12Ø30, TR: (4)Ø10/10 cm	0.65×0.65, LR:8Ø22+ 4Ø26, TR: (4)Ø10/10 cm	0.55×0.55, LR:8Ø18+ 4Ø22, TR: (2)Ø10/20 cm	0.60×0.55, LR:8Ø20+ 4Ø24, TR: (2)Ø10/20 cm
	h10×b10	0.75×0.75, LR: 28Ø30, TR: (4)Ø10/10 cm	0.65×0.65, LR:8Ø22+ 12Ø26, TR: (4)Ø10/10 cm	0.55×0.50, LR:6Ø20+ 12Ø28, TR: (2)Ø10/20 cm	0.50×0.35, LR:4Ø26+ 4Ø32, TR: (2)Ø10/20 cm
	h11×b11	0.75×0.75, LR: 28Ø30, TR: (4)Ø10/10 cm	0.65×0.65, LR:8Ø26+ 12Ø32, TR: (4)Ø10/10 cm	0.50×0.50, LR:4Ø22+ 12Ø26, TR: (2)Ø10/20 cm	0.45×0.45, LR:4Ø22+ 12Ø26, TR: (2)Ø10/20 cm
	h12×b12	0.80×0.80, LR: 34Ø30, TR: (4)Ø10/10 cm	0.65×0.65, LR:8Ø26+ 12Ø32, TR: (4)Ø10/10 cm	0.55×0.55, LR:8Ø22+ 12Ø26, TR: (2)Ø10/20 cm	0.55×0.55, LR:8Ø24+ 4Ø28, TR: (2)Ø10/20 cm
Beams	h13×b13	0.60×0.60, LR: 26Ø20+ 35Ø18, TR: (2)Ø8/15 cm	0.65×0.65, LR:31Ø20, TR: (2)Ø8/15 cm	0.55×0.55, LR:11Ø18+ 10Ø20, TR: (2)Ø8/15 cm	0.60×0.55, LR:6Ø18+ 5Ø20, TR: (2)Ø8/15 cm

Table 7 continued

	Optimum designs			
	q = 1	q = 2	q = 3	q = 4
h14×b14	0.75×0.75, LR: 26Ø20+ 35Ø18, TR: (2)Ø8/15 cm	0.65×0.65, LR:33Ø20, TR: (2)Ø8/15 cm	0.55×0.50, LR:9Ø18+ 10Ø20, TR: (2)Ø8/15 cm	0.50×0.35, LR:6Ø18+ 3Ø16, TR: (2)Ø8/15 cm
$C_{IN,RC\,Frame}$(1,000 €)	3.92E+02	3.51E+02	2.40E+02	1.99E+02
C_{IN}(1,000 €)	1.59E+03	1.55E+03	1.44E+03	1.40E+03

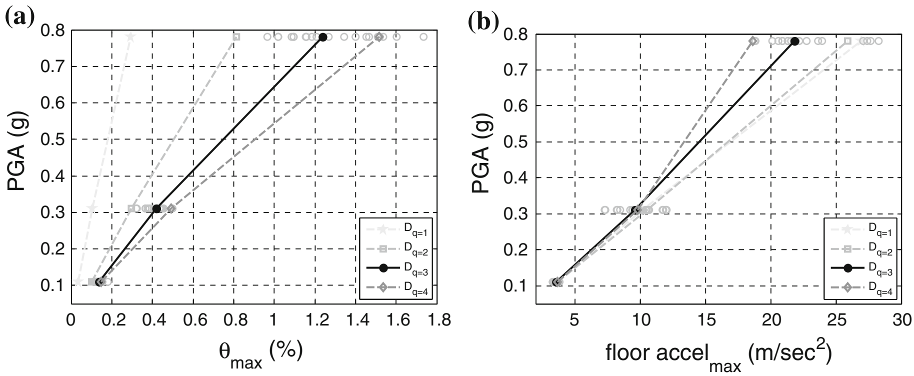

Fig. 3 Five storey test example—initial (CIN), expected (CLC) and total expected (TOT) life-cycle costs for different values of the behaviour factor q ($t = 50$ years, $\lambda = 5\%$)

second dominant contributors. It is worth mentioning, that the contribution of the cost of contents due to floor acceleration on the limit-state cost is only 20% for design $D_{q=4}$ while it is almost 85% for design $D_{q=1}$. This is due to the fact that the latter design is much stiffer and thus increased floor accelerations inflict significant damages on the contents. It has also to be noticed that although the four designs differ significantly, injury and fatality costs represent only a small quantity of the total cost: 0.015% for design $D_{q=1}$, while for designs $D_{q=2}$, $D_{q=3}$ and $D_{q=4}$ represents the 0.25, 1.0 and 2.3% of the total cost, respectively.

7.2 Eight storey symmetrical test example

The plan and front views of the eight storey symmetrical test example are shown in Fig. 6. The structural elements (beams and columns) are separated into 14 groups, 12 groups for the columns and 2 for the beams, resulting into 70 design variables. The optimum designs achieved for different values of the q factor are presented in Table 7. It can be seen that, with respect to total initial cost design $D_{q=1}$ is increased by the marginal quantity of 3% compared to $D_{q=2}$ and by 10 and 15% compared to $D_{q=3}$ and $D_{q=4}$, respectively. In the case when the four designs are compared with reference to the cost of the RC skeletal members alone, design $D_{q=1}$ is increased by 12% compared to $D_{q=2}$ and by 65 and 95% compared to $D_{q=3}$ and $D_{q=4}$, respectively. Confirming the results of the first test example, it can be also

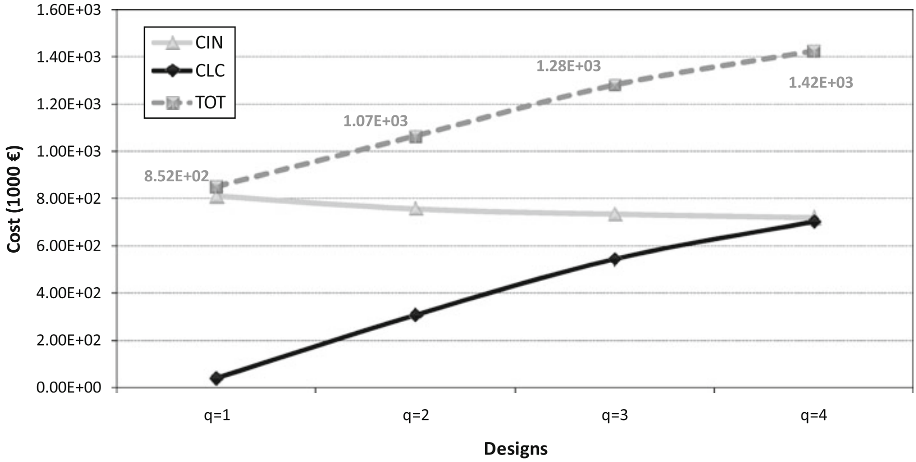

Fig. 4 Five storey test example—initial (CIN), expected (CLC) and total expected (TOT) life-cycle costs for different values of the behaviour factor q ($t = 50$ years, $\lambda = 5\%$)

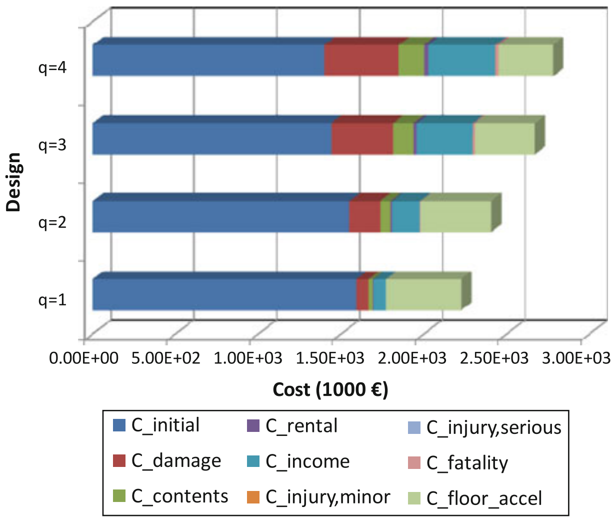

Fig. 5 Five storey test example—contribution of the initial cost and limit state cost components to the total expected life-cycle cost for different values of the behaviour factor q

seen that the initial construction cost of RC structures designed based on elastic response for the design earthquake is by no means prohibitive

The median values of the four cases employed for defining the *abscissa* values of both $(\bar{P}_i - \theta_i)$ and $(\bar{P}_i - \ddot{u}_{floor,i})$ pairs, are shown in Fig. 7a, b, while they have been obtained through 20 non-linear time history analyses performed for each of the three hazard levels (50/50, 10/50 and 2/50). Figure 8 depicts the optimum designs obtained with reference to the four behaviour factors, along with the initial construction, limit state and total life-cycle costs, calculated for the Ghobarah drift limits (Ghobarah 2004). In accordance to the previous test example, a general observation can be obtained from this figure that design $D_{q=4}$ is worst compared to the other three designs with respect to C_{TOT}. Comparing design $D_{q=3}$, obtained

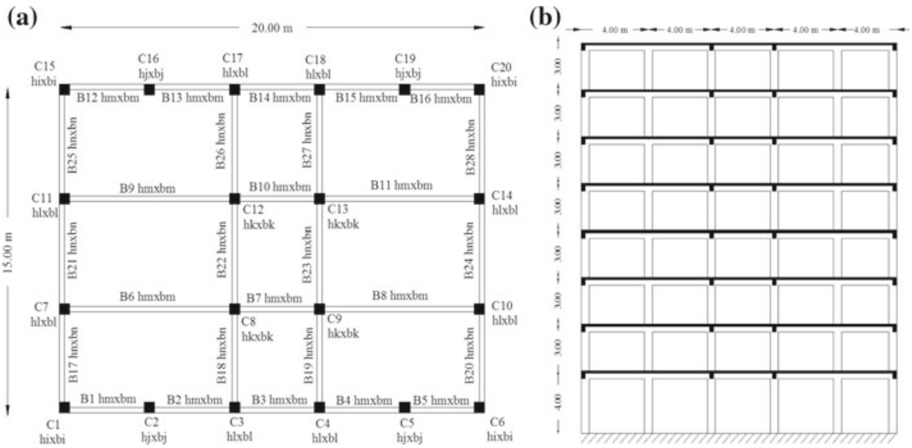

Fig. 6 Eight storey test example—**a** plan view, **b** front view

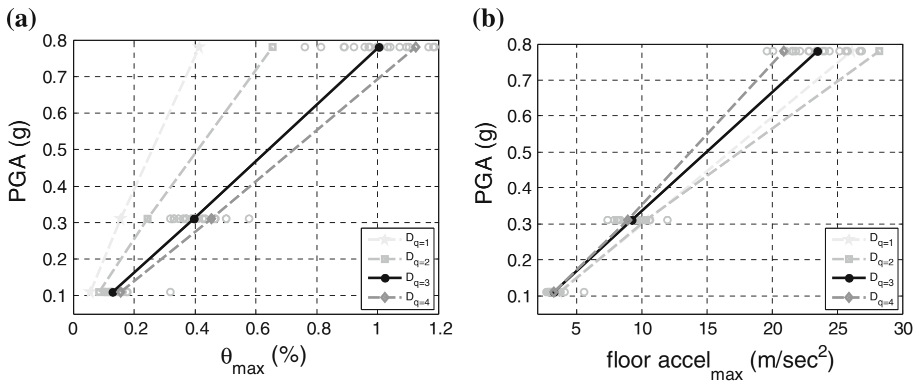

Fig. 7 Eight storey test example—50% median values of the maximum **a** interstorey drift values and **b** floor accelerations for the four designs

for the behaviour factor suggested by the Eurocodes for RC buildings, it can be seen that it is 30 and 17% more expensive compared to $D_{q=1}$ and $D_{q=2}$, respectively; while it is 6%, less expensive with reference to C_{TOT}, compared to $D_{q=4}$.

The contribution of the initial and limit state cost components to the total life-cycle cost are shown in Fig. 9. C_{IN} represents the 72% of the total life-cycle cost for design $D_{q=1}$ while for designs $D_{q=2}$, $D_{q=3}$ and $D_{q=4}$ represents the 65, 54 and 50%, respectively. Although the initial cost is the dominant contributor for all optimum design; for designs $D_{q=1}$ and $D_{q=2}$ the second dominant contributor is the cost of contents due to floor acceleration while for designs $D_{q=3}$ and $D_{q=4}$ damage and income costs are almost equivalent representing the second dominant contributors. It is worth mentioning, as in the previous example, that the contribution of the cost of contents due to floor acceleration on the limit-state cost is only 29%, as in the previous example, for design $D_{q=4}$ while it is almost 76% for design $D_{q=1}$. Furthermore, although the four designs differ significantly injury and fatality costs represent only a small quantity of the total cost: 0.041% for design $D_{q=1}$, while for designs $D_{q=2}$, $D_{q=3}$ and $D_{q=4}$ represents the 0.14, 0.46 and 0.68% of the total cost, respectively.

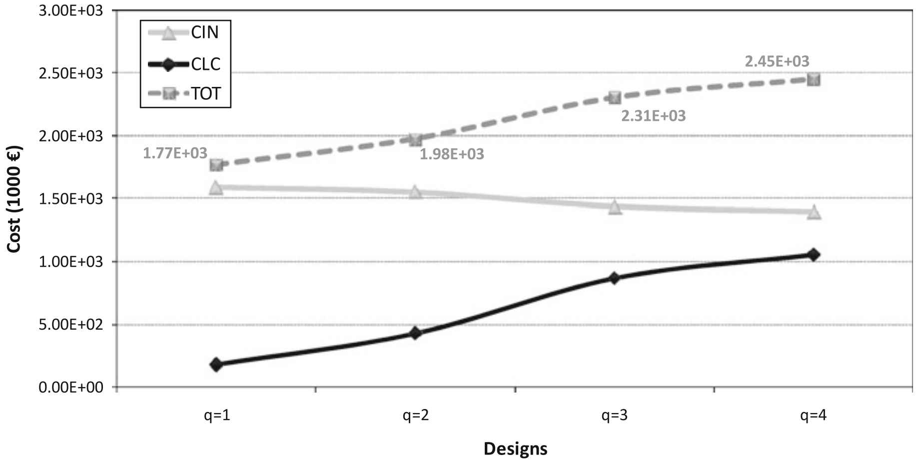

Fig. 8 Eight storey test example—initial (CIN), expected (CLC) and total expected (TOT) life-cycle costs for different values of the behaviour factor q ($t = 0$ years, $\lambda = 5\%$)

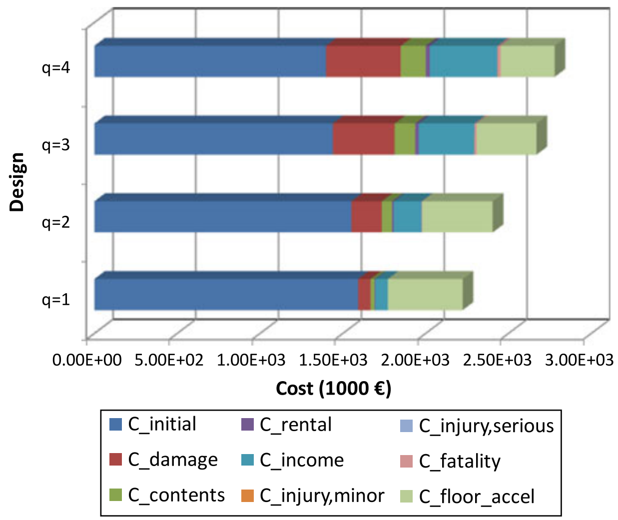

Fig. 9 Eight storey test example—Contribution of the initial cost and limit state cost components to the total expected life-cycle cost for different values of the behaviour factor q

8 Concluding remarks

An investigation was performed on the effect of the behaviour factor q in the final design of reinforced concrete buildings under earthquake loading in terms of safety and economy. The numerical tests were performed on two multi-storey reinforced concrete buildings having symmetrical and non symmetrical plan views which were optimally designed according the European codes EC2 and EC8. The values of the behaviour factor varied from $q = 1$, representing the elastic response, to $q = 4$. Although the two test examples are different this study resulted in quite similar findings for the two cases considered.

The main findings of this study can be summarized in the following:

Table 8 Eight storey test example-limit state cost components (1,000 €)

Limit state	C^i_{dam}	C^i_{con}	C^i_{ren}	C^i_{inc}	C^i_{inj}		C^i_{fat}	$C^i_{LS}(I)$	$C^i_{LS}(II)$
					Minor	Serious			
I	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
II	18.00	6.00	2.59	43.20	0.00	0.00	0.13	69.79	69.93
III	180.00	60.00	9.59	159.84	0.03	0.04	1.34	409.43	410.84
IV	720.00	240.00	35.71	595.20	0.29	0.38	13.44	1590.91	1605.02
V	1620.00	540.00	100.22	1670.40	2.88	3.84	134.40	3930.62	4071.74
VI	2880.00	960.00	188.35	3139.20	28.80	38.40	1344.00	7167.55	8578.75
VII	3600.00	1200.00	288.00	4800.00	38.40	384.00	26880.00	9888.00	37190.40

1. The initial cost of reinforced concrete structures designed based on elastic response $D_{q=1}$ is not excessive since it varies, for the two representative test cases considered, from 3 to 15% compared to the initial cost of the designs $D_{q=2}$ to $D_{q=4}$, respectively. In fact, the designs $D_{q=1}$ are only by 10% more expensive compared to the cost of the designs obtained for the value of the behaviour factor suggested by the Eurocode (q = 3). In the case, though, that the four designs are compared with reference to the cost of the RC skeletal members alone, design $D_{q=1}$ is 95% more expensive compared to D_q (q = 2,3,4).
2. Comparing, the contributing parts (Figs. 5 and 9) of the total life-cycle cost, it can be said that the initial cost, in both test examples, is the first dominant contributor for all designs obtained. The cost of contents due to floor acceleration is the second dominant contributor for stiffer design ($D_{q=1}$ and $D_{q=2}$), while for designs $D_{q=3}$ and $D_{q=4}$ damage and income costs are almost equivalent representing the second dominant contributors. It is also worth mentioning that, although the four designs differ significantly, injury and fatality costs represent only a small percentage of the total cost.
3. The contribution of the cost of contents due to floor acceleration on the limit-state cost was in the range 20–29% for design $D_{q=4}$ while it was found in the range 76–85% for design $D_{q=1}$. This is due to the fact that the latter design is much stiffer compared to the other ones and thus increased floor accelerations inflict significant damages on the contents.
4. The main conclusion that can be drawn from this study is that the behavior factor suggested by the Eurocode does not lead to the most safe and economic design.

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References

ATC-13 (1985) Earthquake Damage Evaluation Data for California. Applied Technology Council, Redwood City

ATC-19 (1995) Structural response modification factors. Applied Technology Council, Redwood City

ATC-34 (1995) A critical review of current approaches to earthquake resistance design. Applied Technology Council, Redwood City

Borzi B, Elnashai AS (2000) Refined force reduction factors for seismic design. Eng Struct 22(10):1244–1260

Chan C-M, Zou X-K (2004) Elastic and inelastic drift performance optimization for reinforced concrete buildings under earthquake loads. Earthquake Eng Struct Dyn 33(8):929–950

Cuesta I, Aschheim MA, Fajfar P (2003) Simplified R-factor relationships for strong ground motions. Earthquake Spectra 19(1):25–45
EN 1998-1:2003 (2003) Eurocode 8: Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings. Commission of the European Communities, European Committee for Standardization, October
Elenas A, Meskouris K (2001) Correlation study between seismic acceleration parameters and damage indices of structures. Eng Struct 23:698–704
Ellingwood BR, Wen Y-K (2005) Risk-benefit-based design decisions for low-probability/high consequence earthquake events in mid-America. Prog Struct Eng Mater 7(2):56–70
Fajfar P (1998) Towards nonlinear methods for the future seismic codes. In: Booth (Ed.) Seismic design practice into the next century. Balkema
FEMA 227 (1992) A benefit–cost model for the seismic rehabilitation of buildings. Federal Emergency Management Agency. Building Seismic Safety Council, Washington
Frangopol DM (1986) Computer-automated design of structural systems under reliability-based performance constraints. Eng Comput 3(2):109–115
Gasparini DA, Vanmarcke EH (1976) Simulated earthquake motions compatible with prescribed response spectra. MIT civil engineering research report R76-4. Massachusetts Institute of Technology, Cambridge
Ghobarah A, Abou-Elfath H, Biddah A (1999) Response-based damage assessment of structures. Earthquake Eng Struct Dyn 28(1):79–104
Ghobarah A (2004) On drift limits associated with different damage levels. International Workshop on Performance- Based Seismic Design, June 28–July 1
Kappos AJ (1999) Evaluation of behaviour factors on the basis of ductility and overstrength studies. Eng Struct 21(9):823–835
Kang Y-J, Wen YK (2000) Minimum life-cycle cost structural design against natural hazards, Structural Research Series No. 629. Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, Urbana
Karavasilis TL, Bazeos N, Beskos DE (2007) Behaviour factor for performance-based seismic design of plane steel moment resisting frames. J Earthquake Eng 11(4):531–559
Karakostas CZ, Athanassiadou CJ, Kappos AJ, Lekidis VA (2007) Site-dependent design spectra and strength modification factors, based on records from Greece. Soil Dyn Earthquake Eng 27(11):1012–1027
Lagaros ND (2007) Life-cycle cost analysis of construction practices. Bull Earthquake Eng 5:425–442
Lagaros ND, Papadrakakis M (2007) Seismic design of RC structures: a critical assessment in the framework of multi-objective optimization. Earthquake Eng Struct Dyn 36(12):1623–1639
Lagaros ND, Garavelas ATh, Papadrakakis M (2008) Innovative seismic design optimization with reliability constraints. Comput Methods Appl Mech Eng 198(1):28–41
Lam N, Wilsont J, Hutchinson G (1998) The ductility reduction factor in the seismic design of buildings. Earthquake Eng Struct Dyn 27:749–769
Lee K, Foutch DA (2006) Seismic evaluation of steel moment frame buildings designed using different R-values. J Struct Eng 132(9):1461–1472
Li G, Cheng G (2001) Optimal decision for the target value of performance-based structural system reliability. Struct Multidiscip Optim 22(4):261–267
Lu Y, Hao H, Carydis PG, Mouzakis H (2001) Seismic performance of RC frames designed for three different ductility levels. Eng Struct 23(5):537–547
Mazzolani FM, Piluso V (1996) The theory and design of seismic resistant steel frames. E & FN Spon,
Miranda E, Bertero VV (1994) Evaluation of strength reduction factors for earthquake resistant design. Earthquake Spectra 10(2):357–379
Moharrami H, Grierson DE (1993) Computer-automated design of reinforced concrete frameworks. J Struct Eng 119(7):2036–2058
Olsson A, Sandberg G, Dahlblom O (2003) On latin hypercube sampling for structural reliability analysis. Struct Saf 25(1):47–68
Palazzo B, Petti L (1996) Reduction factors for base isolated structures. Comput Struct 60(6):945–956
Papaioannou I, Fragiadakis M, Papadrakakis M (2005) Inelastic analysis of framed structures using the fiber approach, Proceedings of the 5th International Congress on Computational Mechanics (GRACM 05), Limassol, Cyprus, June 29–July 1
Papazachos BC, Papaioannou ChA, Theodulidis NP (1993) Regionalization of seismic hazard in Greece based on seismic sources. Natural Hazards 8(1):1–18
PrEN 1992-1-1:2002 (2002) Eurocode 2: Design of Concrete Structures- Part 1: General Rules and Rules for Buildings. Commission of the European Communities, European Committee for Standardization, December

Rackwitz R (2006) The effect of discounting, different mortality reduction schemes and predictive cohort life tables on risk acceptability criteria. Reliab Eng Syst Saf 91(4):469–484

Rackwitz R, Lentz A, Faber M (2005) Socio-economically sustainable civil engineering infrastructures by optimization. Struct Saf 27(3):187–229

Sarma KC, Adeli H (1998) Cost optimization of concrete structures. J Struct Eng 124(5):570–578

Wen YK, Kang YJ (2001) Minimum building life-cycle cost design criteria. I: Methodology. J Struct Eng 127(3):330–337

Wen YK, Kang YJ (2001) Minimum building life-cycle cost design criteria. II: Applications. J Struct Eng 127(3):338–346

Whittaker A, Hart G, Rojahn C (1999) Seismic response modification factors. J Struct Eng 125(4):438–444