ECCOMAS Congress 2016
VII European Congress on Computational Methods in Applied Sciences and Engineering
M. Papadrakakis, V. Papadopoulos, G. Stefanou, V. Plevris (eds.)
Crete Island, Greece, 5–10 June 2016

# OPTIMUM DESIGN OF REINFORCED CONCRETE FRAMES ACCORDING TO EC8 AND MC2010 WITH GENETIC ALGORITHMS

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**Keywords:** Reinforced concrete, seismic, performance-based, Eurocodes, fib Model Code 2010, optimization, genetic-algorithms.

**Abstract.** *The need for cost-efficient seismic design in limited time has led to the development of automated structural optimization methodologies. Genetic Algorithms (GA) belong to the class of stochastic, nature-inspired heuristic optimization algorithms. GA can be easily implemented and applied to advanced structural problems since they don't require use of gradients of cost or constraints functions. Furthermore, they are able to identify global optima as opposed to local optimum solutions. Early efforts to optimise seismic design of concrete structures were based on traditional seismic design code approaches. The last two decades, performance- and deformation-based seismic design methodologies have emerged. These methodologies provide enhanced structural damage control for different earthquake intensities reducing both economic losses and human casualties. Lately, a fully-fledged performance- and deformation-based design methodology has been incorporated in the fib Model Code for Concrete Structures MC2010, which is meant to serve as a guidance document for future design codes of concrete structures. To the best of author's knowledge, there exists no study investigating optimum seismic design solutions in accordance with MC2010. The aim of this study is to develop a GA-based seismic design optimization framework for reinforced concrete frames in accordance with the provisions of both Eurocode 8 (EC8) and MC2010. Application of this framework to reinforced concrete frames is conducted and comparisons of the optimum solutions obtained by the two seismic design guidelines are made. The advantages and disadvantages of the two seismic design methodologies, in the context of structural optimization, are highlighted and discussed.*

## 1 INTRODUCTION

The need for improved control of structural damage for different levels of seismic action has led to the development of performance-based seismic design. Performance-based seismic design is a transparent and direct design framework that requires a set of performance levels to be met for different levels of seismic hazard. Performance levels are related to the level of structural damage of the structure, which in turn is directly related to structural member deformations and/or inter-story drifts.

The new *fib* Model Code 2010 (MC2010) includes a fully-fledged performance-based seismic design and assessment methodology for various levels of seismic hazard (*fib* 2010, Fardis 2013). MC2010 will serve as a basis for future codes for concrete structures.

In MC2010, each performance limit state corresponds to a specific physical condition of the structure and it is expressed in terms of deformation limits of the structural members providing direct control of allowable seismic damage. The levels of seismic hazard are identified by their annual probability of being exceeded. Seismic actions are specified in terms of acceleration time-histories of the ground motion components. The reference method for determining seismic demands is the most rigorous inelastic response history analysis with step-by-step integration of the equation of motion in the time domain (Fardis 2013).

Extensive research has been conducted over the past decades on optimum seismic design of structures (Fragiadakis and Lagaros 2011). However, only a small part of this research has been dedicated to reinforced concrete structures. This can be partially attributed to the complex nature and detailing of reinforced concrete structures that increases significantly the number of design variables (Sarma and Adeli 1998).

The number of research studies on optimization of performance and deformation-based seismic design of reinforced concrete structures is limited. Ganzerli *et al*. (2000) were the first, to the best of the author's knowledge, to consider seismic optimization with performance-based constraints. Chan and Zou (2004) examined optimum seismic design of reinforced concrete frames by employing optimality criteria approach. Lagaros and Papadrakakis (2007) compared the provisions of EC8 for seismic analysis of 3D reinforced concrete structures with a performance-based seismic design methodology in the framework of multi-objective optimization. Fragiadakis and Papadrakakis (2008) presented a performance-based optimum seismic design methodology for reinforced concrete frames based on nonlinear time history analyses. Inter-story drifts were used as performance criteria. Gencturk (2013) investigated performance-based seismic design optimization of reinforced concrete and reinforced engineered cementitious composites (ECC) frames, by using Taboo Search optimization algorithm.

It can be concluded from the above, that no study has been conducted so far on optimization of reinforced concrete frames in accordance with MC2010 seismic design provisions. To address this gap, this study presents optimum seismic design solutions of reinforced concrete frames obtained by MC2010 and compares them with optimum designs following EC8 (CEN 2004) guidelines. To serve this goal, a general computational optimization framework for reinforced concrete frames is developed that makes use of a genetic algorithm able to track global optima of complex problems with discrete design variables.

## 2 OPTIMIZATION OF REINFORCED CONCRETE FRAMES

### 2.1 Optimization framework

In optimization problem formulations, the goal is to minimize an objective function $C(\boldsymbol{x})$ subject to $m$ number of constraints $g_j(\boldsymbol{x})\leq 0$ ($j$=1 to $m$). A design solution is represented by the design vector $\boldsymbol{x}$, which contains $n$ number of independent design variables $x_i$ ($i$=1 to $n$). In deterministic structural seismic design optimization, the objective function $C(\boldsymbol{x})$ is typically the initial cost of the structure. Constraints $g_j$ are either related to engineering demand parameters (*EDP*) (e.g. forces, displacements, rotations, drifts, etc.) or to detailing rules set by design codes and construction practice. To realistically represent construction practice, design variables $x_i$ typically take values from discrete sets of values $\boldsymbol{D}_i=(d_{i1}, d_{i2}, \ldots, d_{iki})$, where $d_{ip}$ ($p$=1 to $k_i$) is the $p$-th possible discrete value of design variable $x_i$ and $k_i$ is the number of allowable discrete values of $x_i$. For reinforced concrete structures, design variables are generally related to concrete section dimensions and steel reinforcement.

### 2.2 Genetic algorithm (GA)

Genetic Algorithm (GA) (Holland 1975) belongs to the class of stochastic, nature-inspired heuristic algorithms. It is based on Darwin's theory of natural selection and evolution. Genetic algorithm can be easily implemented and applied to advanced optimization problems since it doesn't require use of gradients of cost or constraints functions. Furthermore, it is able to identify global optima as opposed to local optimum solutions.

The GA iteratively modifies populations (generations) of individuals in order to evolve toward an optimum solution. An individual $\boldsymbol{x}$ (genome) represents a candidate solution to the optimization problem. The values of the design variables $x_i$ ($i$=1 to n) forming each individual are called genes. Furthermore, the objective function of each individual is known as fitness function. The best fitness value of a generation is the smallest fitness value of all individuals of the generation. In order to create the next population, the genetic algorithm selects certain individuals in the current population (parents) and uses them to create individuals in the next generation (children).

In this study, the mixed integer GA as implemented in MATLAB-R2015a (MathWorks 2015) is employed. This algorithm can handle both continuous and discrete design variables. The algorithm is able to account for nonlinear constraints by using the penalty function approach. The genetic algorithm in this study is terminated when one of the following stopping criteria is met:

i) Number of generations exceeds a pre-specified maximum number of generations.

ii) The mean relative variation of the best fitness value does not exceed a pre-specified tolerance over a pre-specified number of generations.

### 2.3 Design variables

As shown in Fig. 1, design variables are separated in column and beam section design variable sub-vectors. Assembly of these sub-vectors forms the design variables vector $\boldsymbol{x}$.

Column design variable sub-vectors are the heights $\boldsymbol{h_c}$ and widths $\boldsymbol{b_c}$ of the column sections, the diameters $\boldsymbol{d_{bc}}$ and numbers of main bars per side $\boldsymbol{n_c}$, assumed the same for all column section sides for simplicity, the diameters $\boldsymbol{d_{bwc}}$, spacings $\boldsymbol{s_c}$ and numbers of legs $\boldsymbol{n_{wc}}$ of transverse reinforcement assumed again the same in both column section directions for simplification purposes.

Beam design variable sub-vectors are the heights $\boldsymbol{h_b}$ and widths $\boldsymbol{b_b}$ of the beam sections, the diameters $\boldsymbol{d_{bt}}$ and numbers of main bars of the top sides $\boldsymbol{n_{tb}}$, the diameters $\boldsymbol{d_{bb}}$ and numbers of main bars of the bottom sides $\boldsymbol{n_{bb}}$, the diameters $\boldsymbol{d_{bwb}}$, spacings $\boldsymbol{s_b}$ and numbers of legs $\boldsymbol{n_{wb}}$ of transverse reinforcement parallel to beam section heights.

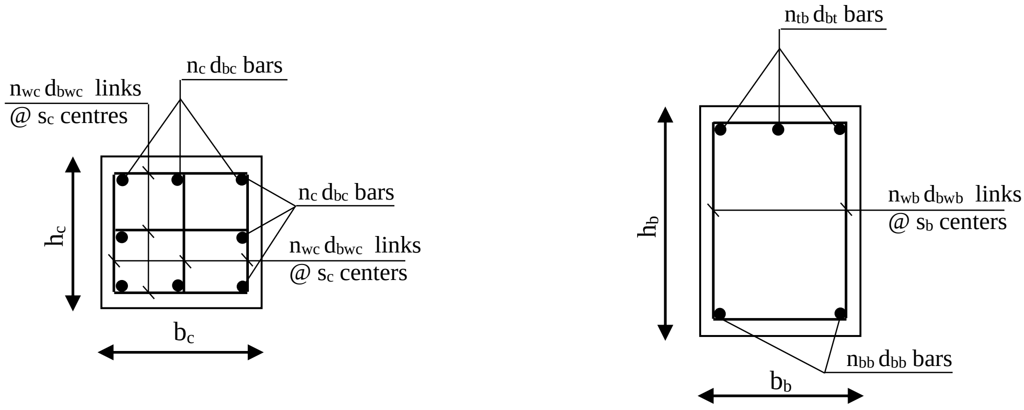



Figure 1: Design variables: a) column sections, b) beam sections

### 2.4 Objective function

In this study, the objective function $C(\boldsymbol{x})$ is the initial or construction cost of the reinforced concrete frames. The construction cost consists of the cost of the materials and the cost of the formwork of beam and column members. The following unit costs are assumed for concrete: 100Euros/m$^3$, steel: 1Euro/kg and formwork: 15Euros/m$^2$.

### 2.5 Design constraints

In seismic design of reinforced concrete frames, constraints $g_j(\boldsymbol{x})$ are either related to engineering demand parameters (*EDP*) (e.g. forces, displacements, rotations, drifts, etc.) or to detailing rules set by design codes and construction practice. In the first case, an *EDP* must remain below a limit value $EDP_{cap}$. This type of constraints can be written in the following normalized form

$$EDP \leq EDP_{cap} \rightarrow \frac{EDP}{EDP_{cap}} - 1 \leq 0 \tag{1}$$

Regarding detailing requirements, the constraints can be expressed in terms of structural design parameters *SDP*. It is noted that a *SDP* can be a design variable itself (e.g. column height, main bar diameter) or a simple function of the design variables like the volumetric ratios of steel reinforcement.

In some cases, it is required that a *SDP* remains lower than or equal to a maximum value $SDP_{max}$. This category of constraints is written in the following general form:

$$SDP \leq SDP_{max} \rightarrow \frac{SDP}{SDP_{max}} - 1 \leq 0 \tag{2}$$

In other cases, it is required that a *SDP* is greater than or equal to a minimum value $SDP_{min}$. The latter family of constraints is expressed in the normalized form shown below:

$$SDP \geq SDP_{min} \rightarrow \frac{SDP_{min}}{SDP} - 1 \leq 0 \tag{3}$$

## 3 OPTIMUM DESIGN ACCORDING TO EUROCODE 2

Prior to designing for seismic actions, RC frames must be designed to resist dead and live loads. Eurocode 2 (EC2 – CEN 2004) provisions are applied in this study for designing against static loads. EC2 provisions consist of a number of detailing rules and a number of requirements related to *EDPs*.

Regarding detailing rules, design constraints of minimum volumetric ratio of longitudinal reinforcement, minimum diameter of longitudinal and transverse reinforcement, minimum distance between two longitudinal steel bars and minimum volumetric ratio of transverse reinforcement are expressed in the general form of Eq. (3). On the other hand, constraints of the maximum volumetric ratio of longitudinal reinforcement, maximum spacing of shear reinforcement and maximum distance of unrestrained next to restrained main bars of columns are written in the form of Eq. (2).

For the ULS, *EDPs* are member forces (moments and shear forces) derived by linear elastic analysis for the following load combination, where $G_k$ represents the characteristic value of the permanent action and $Q_k$ stands for the characteristic value of the variable action.

$$S_d = 1.35G_k + 1.50Q_k \tag{4}$$

*EDPs* constraints are written in the general form of Eq. (1), where capacities $R_d$ are derived by using characteristic material strengths divided by partial safety factors equal to $\gamma_c$=1.50 for concrete and $\gamma_s$=1.15 for reinforcing steel. For bending moments of column members, moment capacities are calculated for the axial load demand of the load combination under examination.

For beam deflections, the limiting span to depth ratio approach is used herein ensuring that deflections are limited to span/250. Moreover, crack control is achieved by limiting maximum bar size or spacing.

## 4 OPTIMUM SEISMIC DESIGN ACCORDING TO EUROCODE 8

Seismic design according to EC8 can be performed either without provisions for energy dissipation and ductility (Ductility Class Low – DCL) or with provisions for energy dissipation and ductility (Ductility Classes Medium and High – DCM and DCH). DCM and DCH differ in the levels of lateral strength and allowable inelastic response. DCH allows for further reductions in seismic forces, but requires more demanding prescriptive rules for increasing ductility capacities than DCM.

For DCL, all seismic *EDPs* are calculated from the seismic load combination shown below, where design seismic actions $E_d$ are calculated by the design response spectrum that is derived from the elastic response spectrum reduced by the behaviour factor $q$. $\psi_2$ is the quasi-permanent load combination coefficient of the variable action. Reference analysis method of EC8 is the modal response spectrum analysis. However, for regular buildings with unimportant higher modes the linear elastic lateral force method can also be applied.

$$S_{Ed} = G_k + \psi_2 \cdot Q_k + E_d \tag{5}$$

For DCM and DCH, first the dissipative zones of structural members (typically located at the ends) are designed in bending under the seismic design load combination. Next, capacity design principles are forced to ensure ductile structural response. In particular, column sections are designed in bending following the strong column – weak beam capacity rule to prevent soft storey failure mechanisms. Moreover, capacity design in shear is applied to beam and column members and joints to preclude brittle shear failures.

In addition to the above, RC frames are checked for a 'frequent' earthquake with 10% probability of exceedance in 10 years (10/10) to satisfy the Damage Limitation (DL) limit state. Checks verify that interstorey drifts developed for the 'frequent' earthquake are less than limit values depending on the type of non-structural elements (e.g. 1% for non-structural elements that don't interfere with structural response).

P-delta ($2^{nd}$ order) effects are considered at the $i$ storey level with calculating ratio $\theta^i$from Eq. (6). In this equation, $N_{tot}^i$ and $V_{tot}^i$ are the total vertical and shear load at the storey respectively, $\Delta\delta^i$is interstorey drift and $H^i$is storey height. It is required that $\theta^i$never exceeds 0.2. Furthermore, if $\theta^i$exceeds 0.1 then $2^{nd}$ order effects are taken into account by multiplying $1^{st}$ order effects by the magnification factor 1/(1-$\theta^i$).

$$\theta^i = \frac{N_{tot}^i \cdot \Delta\delta^i}{V_{tot}^i \cdot H^i} \tag{6}$$

All previous requirements are regarded as *EDPs* constraints and are included in the optimization problem in the general form of Eq. (1). The *EDPs* are member bending moments and shear forces, interstorey drifts and $\theta^i$ ratios. Apart from *EDPs* constraints and EC2 detailing rules, DCM and DCH necessitate additional or stricter detailing rules in the critical regions to accommodate local ductility demands.

The additional column constraints of minimum cross-section sides, minimum volumetric ratio of longitudinal reinforcement, minimum diameter of transverse reinforcement, minimum number of bars per side, and minimum confinement of transverse reinforcement in critical regions are expressed in the general form of Eq. (3). The same holds for the additional beam constraints in critical regions such as minimum volumetric ratio of longitudinal reinforcement, minimum longitudinal bar diameter for DCH, minimum bottom reinforcement at the supports and minimum longitudinal bar diameters crossing interior or exterior joints.

On the other hand, the more demanding column constraints in critical regions for maximum spacing between restrained main bars and spacing of transverse reinforcement are formulated in accordance with Eq. (2). The same holds for the beam constraints of maximum longitudinal reinforcement volumetric ratio and spacing of transverse reinforcement in the locations of the critical regions.

## 5 OPTIMUM SEISMIC DESIGN ACCORDING TO FIB MODEL CODE 2010

*fib* MC2010 adopts a fully-fledged performance-based seismic design methodology (Fardis 2013). The code employs deformation limits, which are directly related to seismic damage, in order to verify 4 district Limit States. The Operational (OP) and Immediate Use (IU) Limit States are related to serviceability of structures, whilst the Life Safety (LS) and Collapse Prevention (CP) are related to loss of lives and structural collapse (Ultimate Limit States ULS). Limit States are checked for different levels of Seismic Hazard. Deformation limits controlling Limit States and corresponding levels of Seismic Hazard recommended by *fib* MC2010 for ordinary structures are listed in Table 1 (Fardis 2013).

The verification of Limit States entails comparisons of chord rotation demands $\theta_{Ed}$ at member ends with yield chord rotations $\theta_y$ at the same locations for the OP Limit State and twice $\theta_y$

for the IU Limit State. Furthermore, the two ULS are checked by comparing the plastic part of chord rotation demands at member ends $\theta^{pl}_{Ed}$ with characteristic values (lower 5% percentile) of the cyclic ultimate plastic hinge rotation capacities $\theta^{pl}_{uk}$ divided by a factor of $\gamma^*_R$=1.35 for the LS Limit State and with $\theta^{pl}_{uk}$ without safety factor for the CP Limit State.

| Limit State | Seismic Hazard | Deformation Limit |
|---|---|---|
| Operational (OP) | Frequent with 70% probability of exceedance in 50 years (70/50) | Mean value of $\theta_y$ |
| Immediate Use (IU) | Occasional with 40% probability of exceedance in 50 years (40/50) | Mean value of $\theta_y$ may be exceeded by a factor of 2.0 |
| Life Safety (LS) | Rare with 10% probability of exceedance in 50 years (10/50) | Safety factor $\gamma^*_R$ of 1.35 against $\theta^{pl}_{u,k}$ |
| Collapse Prevention (CP) | Very rare with 2% probability of exceedance in 50 years (2/50) | $\theta^{pl}_{u,k}$ capacity may be reached ($\gamma^*_R$ =1) |

Table 1: Limit States, Seismic Hazard levels and Deformation Limits recommended by *fib* MC2010 for ordinary structures.

It is recommended (Fardis 2013) that for beams and rectangular columns with ribbed bars yield chord rotation $\theta_y$ is taken from the following equation, where $\varphi_y$ is end section yield curvature, $L_s$ the shear span of the member on the side of the end section, $z$ is lever arm of end section, $a_{scr}$ is a coefficient equal to 1 if shear cracking precedes flexural yielding or equal to 0 if not, $h$ is end section height, $d_{bl}$ and $f_{yl}$ diameter and yield strength of longitudinal reinforcement (MPa) and $f_c$ member concrete strength in MPa.

$$\theta_y = \frac{\varphi_y(Ls + a_{scr} \cdot z)}{3} + 0.0014 \cdot \left(1 + \frac{1.5h}{L_s}\right) + \frac{\varphi_y d_{bL} f_{yl}}{8\sqrt{f_c}} \tag{7}$$

Furthermore, characteristic ultimate plastic hinge rotation capacity $\theta^{pl}_{u,k}$ is derived by the respective mean value $\theta^{pl}_{um}$ divided by safety factor $\gamma_{Rd}$. When $\theta^{pl}_{u,m}$ is calculated by the following empirical relationship $\gamma_{Rd}$ can be taken equal to 1.75.

$$\theta^{pl}_{um} = 0.0143 \cdot 0.25^{v} \cdot f_c^{0.2} \cdot \left(\frac{\max(0.01;\omega_2)}{\max(0.01;\omega_1)}\right)^{0.3} \cdot \left(\min\left(9;\frac{L_s}{h}\right)\right)^{0.35} \cdot 25^{\left(\frac{a\rho_w f_{yw}}{f_c}\right)} \tag{8}$$

In Eq. (8), $\omega_1$ and $\omega_2$ are mechanical ratios of reinforcement in tension and compression zone respectively, $v$ is normalized axial load ratio, $a$ is confinement effectiveness factor and $\rho_w$ and $f_{yw}$ are volumetric ratio and yield strength of transverse reinforcement. It is noted that Eq. (8) is recommended for rectangular beams and columns with ductile steel reinforcement and without diagonal reinforcement.

In addition to chord rotation checks, brittle shear failures are checked in terms of internal shear force demands $V_{Ed}$ and design shear force capacities $V_{Rd}$. $V_{Rd}$ outside plastic hinge regions is calculated as for static loadings. Inside plastic hinge regions, *fib* MC2010 specifies a strut inclination of 45° when plastic rotation $\theta^{pl}$ exceeds $2 \cdot \theta_y$ and 21.8° for elastic response ($\theta^{pl}$=0). Interpolation is allowed for intermediate values of $\theta^{pl}$.

The reference analysis method of *fib* MC2010 is nonlinear response history analysis with step-by-step integration of motion equations in the time domain. The finite element model applied should use realistic estimates of the effective elastic stiffness of concrete members $EI_{eff}$. It is recommended in MC2010 that $EI_{eff}$ of concrete members is taken by the following relationship, where $M_y$ represents member end section yield moment and the other parameters have been defined previously.

$$EI_{eff} = \frac{M_y L_s}{3\theta_y} \tag{9}$$

Lumped plasticity finite elements with bilinear moment-rotation hysteretic models and realistic rules for stiffness degradation during unloading and reloading may be employed to model inelastic response of reinforced concrete members.

It is worth noting that when conducting nonlinear analysis both types of seismic demands (i.e. deformations and forces) are obtained directly by the analytical solution without additional considerations for brittle modes of failure (i.e. capacity design principles).

It is also important to clarify that no additional prescriptive rules, like detailing rules set by EC8 for DCM and DCH, need to be applied when designing in accordance with MC2010 apart from the detailing rules required for designing against static loads.

In MC2010, seismic actions are represented by acceleration time-histories of the ground motions. At least seven ground motions are required to use average response values. All acceleration time histories should be scaled such that their elastic response spectrum is not lower than 90% of the target response spectrum for periods ranging between $0.2 \cdot T$ to $T$, where $T$ is the fundamental period of the structure. As it will be shown later in this study, this requirement set by MC2010 can be very onerous and may lead to important increases in the structural cost. It is reminded that EC8 specifies that the mean spectrum of the set of ground motions and not all spectra shouldn't be less than 90% of the target response spectrum in the same range of periods. It is also noted that prior to designing, $T$ is not known and cannot be estimated with accuracy because it depends on steel reinforcement which affects members' yield moments $M_y$ and consequently effective elastic stiffness $EI_{eff}$ as defined in Eq. (9).

## 6 OPTIMUM SEISMIC DESIGN CASE STUDY

In this section, application of the optimum seismic design methodologies described previously to a simple portal RC plane frame is presented. The frame is of ordinary importance, located in a region of high seismicity and rests on soil class B according to the classification of EC8. The frame is designed for 0.16g, 0.24g and 0.36g peak ground acceleration values for the 10/50 seismic hazard level in order to examine the influence of the level of seismicity on the optimum seismic design solutions. The elastic (target) response spectrum with 5% damping of EC8 determined for these specifications and 0.24g peak ground acceleration is shown in Fig. 2.

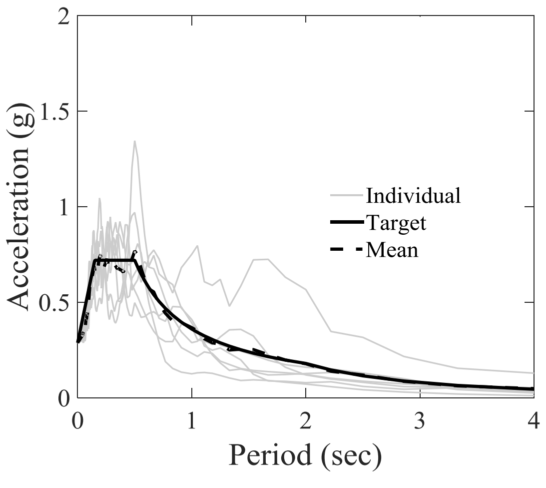
Peak ground accelerations for the other seismic hazard levels of MC2010 objectives are calculated by multiplying by the importance factor $\gamma_I$ given by the following equation proposed in EC8-Part 1, where $P_L$ is the target probability of exceedance in 50 years and $P_{LR}$ is the reference probability of exceedance in 50 years (=10%).

$$\gamma_I = \left(\frac{P_L}{P_{LR}}\right)^{-1/3} \tag{10}$$

The portal frame is designed following the provisions of EC8 for all three ductility classes (i.e. DCL, DCM and DCH) and in accordance with MC2010. In the latter case and in order to evaluate the influence of ground motions selection two different cases are examined. In the first case, designated as THA, the frames are designed for a set of 7 scaled ground motion records satisfying EC8-Part 1 recommendations as described in the previous section. In the second case, designated as THB, the frames are designed for a set of 7 scaled ground motion records satisfying following MC2010 specifications.

Figure 2a presents the scaled and mean elastic spectra with 5% damping of the set of 7 ground motions selected and scaled following EC8 provisions. Selection and scaling was achieved by employing computer program REXEL (Iervolino *et al*. 2009). Because the fundamental period of the structures is unknown prior to their design it was decided to match the mean and target spectrum for periods between 0.1s and 4s in order to capture most possible solutions. The selected ground motion records are all recorded on soil type B and have magnitude $M_w$>5.5. It is evident that the mean spectrum follows very closely the target spectrum.

No computer tools exist for selecting record sets according to MC2010 guidelines. To serve this goal, in this study, a simplified procedure is applied. All records of the European Strong Motion Database (Ambraseys *et al*. 2004) on soil type B with $M_w$>5.5 are scaled so that their scaled 5% damping spectra are not less than 90% of the target spectrum in periods range between 0.1s and 4s. The scaled spectra are later ranked in accordance with their “goodness-of-fit” to the target spectrum as quantified by the normalized root-mean-square-error (Katsanos and Sextos 2013). The first 7 ground motions comprise the set of records used herein. Figure 2b presents the scaled and mean elastic spectra with 5% damping of the set of 7 ground motions selected and scaled following MC2010. It can be seen that the mean spectrum importantly exceeds the target spectrum leading to serious overestimation of seismic demands. Obviously, more advanced selection methods can produce closer mean and target spectra. This set is used herein simply to illustrate the importance of ground motion selection on the optimum design cost of RC frames.



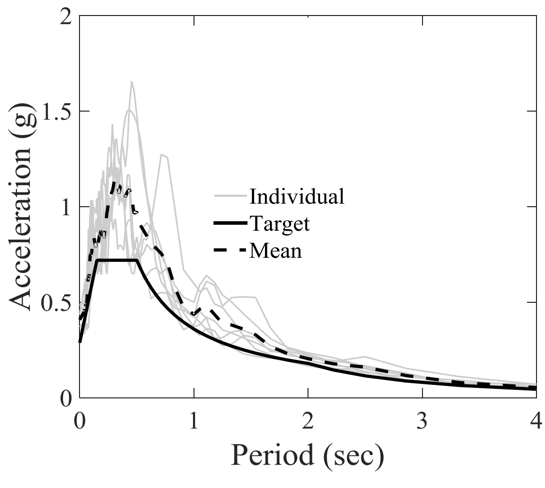




Figure 2: Elastic spectra with 5% damping for ground motion sets selected and scaled in accordance with a) EC8, b) MC2010

For the optimum designs, it is assumed that section dimensions $\boldsymbol{h_c}$, $\boldsymbol{b_c}$, $\boldsymbol{h_b}$, $\boldsymbol{b_b}$ take values from the following discrete set: (0.25m; 0.30m; 0.40m; … ; 1.0m). Furthermore, longitudinal bars $\boldsymbol{d_{bc}}$, $\boldsymbol{d_{bb}}$, and $\boldsymbol{d_{bt}}$ are defined in the following discrete values set: (12mm; 16mm; 20mm; 25mm). Transversal bars $\boldsymbol{d_{bwc}}$, and $\boldsymbol{d_{bwb}}$ take values from: (8mm; 10mm; 12mm). Transverse reinforcement spacing $\boldsymbol{s_c}$ and/or $\boldsymbol{s_b}$ may take the following values: (0.1m; 0.15m; 0.20m; 0.25m; 0.30m).

Finally, numbers of main bars $\boldsymbol{n_c}$, $\boldsymbol{n_{tb}}$, $\boldsymbol{n_{bb}}$ and legs of shear reinforcement $\boldsymbol{n_{wc}}$ and $\boldsymbol{n_{wb}}$ may take any integer value greater than one.

The portal reinforced concrete frame optimally designed herein is shown in Figure 3. The span of the frame is 4m and the height 3m. Concrete C25/30 and reinforcing steel B500C in accordance with EC2 specifications are used. Concrete cover is assumed to be 30mm. Vertical symmetric concentrated loads are applied at the column locations equal to 120.0kN for the permanent and 80.0kN for the live loading. Storey mass for the seismic combination is 29.4t.

The frame consists of two columns C1 and C2 and one beam B1. Due to symmetry, it is assumed that C1 and C2 have exactly the same sections and reinforcement. Concrete members are represented by three sections. Two at the ends to represent critical regions and one to represent the rest part of the element. Due to construction reasons, it is assumed that all sections have the same longitudinal reinforcement. Furthermore, it is assumed because of symmetry that member end sections have the same transverse steel reinforcement. Thus, the same sections are assumed for member ends. Intermediate and end sections differ in the spacing of transverse reinforcement in order to account for the additional requirements in the member critical regions. In total, 18 (8 for columns and 10 for the beam) independent design variables are used in this problem.

The results presented in the following were obtained by running GA with populations of 75 individuals. Iterations were terminated when the mean relative variation of the best fitness value was negligible for 100 generations. MATLAB-R2015a default options were used for GA operations. Furthermore, a significant number of different GA runs for each design solution were conducted and the minimum cost obtained is reported herein.

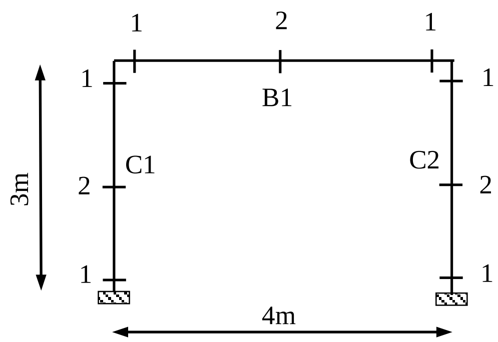



Figure 3: Example portal frame

Figure 4 presents optimization histories of the designs obtained by MC2010 methodology for the THA ground motion set and the three design peak ground accelerations. It can be seen that, as expected, optimum cost increases as design accelerations increase.

Figure 5 compares optimum costs obtained by all seismic design methodologies for the three design peak ground accelerations of the 10/50 seismic hazard level. It can be seen that in all cases costs increase as design accelerations increase. Designs according to EC8 DCL and DCM yield similar costs for all design PGA values. On the other, DCH yields significantly increased costs. This occurred because of the enhanced detailing rules of this ductility class and the discrete design variable sets assumed in this study. This observation shows the influence of detailing requirements on the final costs of reinforced concrete structures.

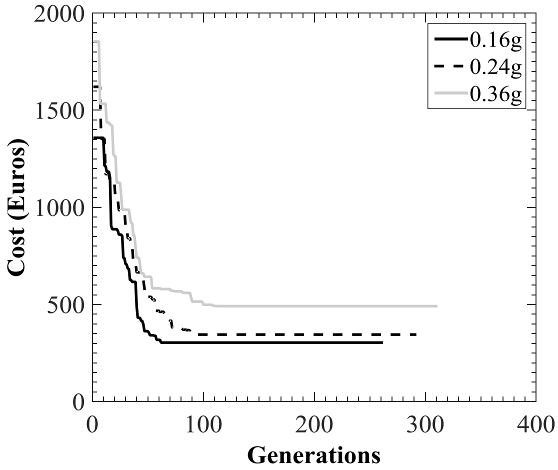



Figure 4: Optimization histories of designs obtained by MC2010 methodology for THA ground motion set and three different design PGAs for the 10/50 seismic hazard level

Furthermore, the direct comparison of optimum costs obtained by MC2010 methodology for the THA and THB ground motion sets shows the importance of the applied accelerograms set. For 0.16g PGA both solutions yield same optimum costs. This is because the design in this case is controlled by minimum detailing requirements. However, for larger seismicity levels the increase in cost by selecting a ground motion data set in accordance with MC2010 provisions is very important.

It is also evident that designs obtained by the MC2010 for both ground motion sets (THA and THB) drive to significantly reduced design costs for the low 0.16g and moderate 0.24g design accelerations. The MC2010 design with THA motion set yields slightly smaller cost than the EC8 designs according to DCL and DCM approaches for 0.36g. However, the same design methodology with the THB motion set drives to significantly greater design costs than all EC8 designs obtained for 0.36g.

Figure 6 presents MC2010 checks of rotation demand constraints for the THA ground motion set and for all Limit States performed for the optimum solutions obtained by all design methodologies for 0.36g. Column sections locations are defined by the column member number (e.g. C1) and a letter designating the location of the section in the member (i.e. B=bottom and T=top). Similarly, beam sections are defined by the beam member number (e.g. B1) and a letter designating the location of the section in the member (i.e. L=left and R=right). Limit States are written with the acronyms shown in Table 1.

It can be concluded that all design solutions perform rather well. DCM and DCH designs do not satisfy beam rotation constraints for the OP Limit State. It is recalled that EC8 does not have any provisions for the OP Limit State. Furthermore, it can be seen that the MC2010 design for THA motion set marginally satisfies beam and column rotation constraints at the OP Limit State and column rotation constraints at the CP Limit State. This shows that these where the controlling (active) constraints of this design. It is also evident that MC2010 design for THB motion satisfies all constraints with a high level of conservatism.

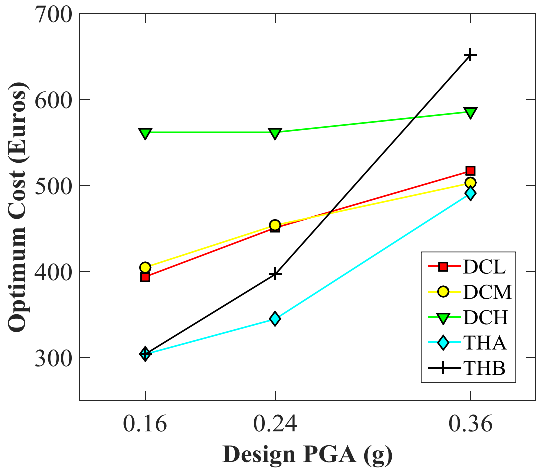



Figure 5: Optimum costs obtained by different design methodologies and design PGAs for the 10/50 seismic hazard level.

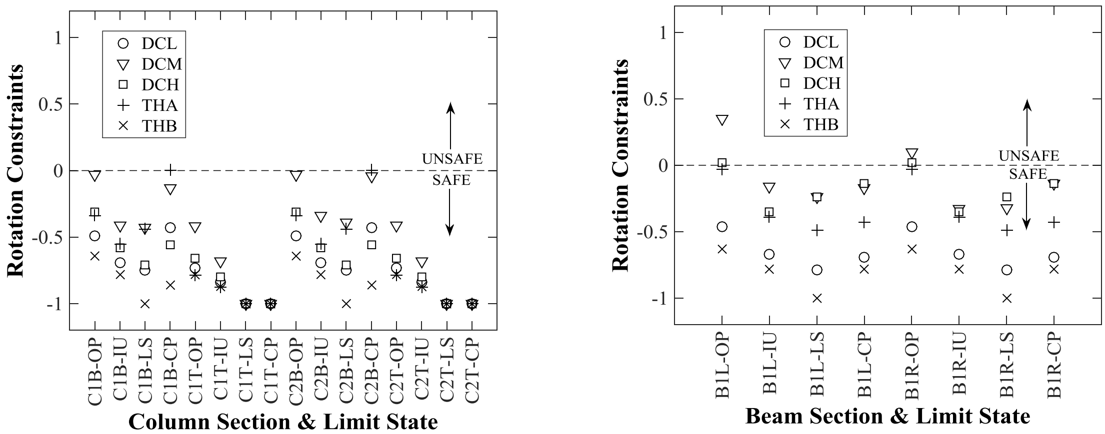



Figure 6: MC2010 rotation constraints of beam and column section optimum solutions obtained by different design methodologies for 0.36g 10/50 seismic hazard PGA.

## 7 CONCLUSIONS

- The new *fib* Model Code 2010 (MC2010) includes a fully-fledged performance-based seismic design and assessment methodology for various levels of seismic hazard. MC2010 will serve as a basis for future codes for concrete structures.
- The reference analysis method of *fib* MC2010 is nonlinear response history analysis with step-by-step integration of motion equations in the time domain. Due to the complexity of this approach, genetic algorithms are well suited to this design methodology since they don’t require use of gradients of cost or constraints functions. Furthermore, they are able to identify global optima as opposed to local optimum solutions.
- This study presents optimum seismic design solutions of a portal reinforced concrete frame obtained by MC2010 and compares them with optimum designs following EC8 (CEN 2004) guidelines for all three ductility classes. The frame is designed for three different peak ground accelerations (0.16g, 0.24g and 0.36g) for the 10/50 seismic hazard level.

- It is found that the construction cost increases with the level of peak ground acceleration for all seismic design methodologies.
- The MC2010 seismic design methodology may lead to serious cost savings for low seismicity levels. However, for high design peak ground accelerations MC2010 and EC8 yield similar optimum costs.
- The selection of the ground motion records set plays an important role on the optimum design costs obtained by the MC2010 approach. If the MC2010 prescriptions are used the design costs increase importantly with respect to the selection of a set of ground motions following EC8 guidelines.
- EC8 optimum design solutions do not satisfy rotation demand constraints set by MC2010 for the OP limit state. This can be attributed to the fact that no explicit provisions exist in EC8 to address this limit state.

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