Optimum seismic design of reinforced concrete frames according to Eurocode 8 and *fib* Model Code 2010

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Abstract. Traditional seismic design, like the one adopted in Eurocode 8 (EC8), is force-based and examining a single level of seismic action. In order to provide improved control of structural damage for different levels of seismic action, the new *fib* Model Code 2010 (MC2010) includes a fully-fledged displacement- and performance-based seismic design methodology. However, the level of complexity and computational effort of the MC2010 methodology is significantly increased. Hence, the use of automated optimization techniques for obtaining cost-effective design solutions becomes appealing if not necessary. This study employs genetic algorithms to derive and compare optimum seismic design solutions of reinforced concrete frames according to EC8 and MC2010. This is important since MC2010 is meant to serve as a basis for future seismic design codes. It is found that MC2010 drives to more cost-effective solutions than EC8 for regions of low seismicity and better or similar costs for regions of moderate seismicity. For high seismicity regions, MC2010 may yield similar or increased structural costs. This depends strongly on the provisions adopted for selecting the set of ground motions. In all cases, MC2010 provides enhanced control of structural damage.

Keywords: Reinforced concrete; seismic design; Eurocodes; *fib* Model Code 2010; optimization; genetic-algorithms

1 Introduction

Seismic design of reinforced concrete frames according to current codes, like Eurocode 8 (EC8) [1], is based on forces. Structural members (i.e. beams and columns) are dimensioned to withstand internal forces at the Ultimate Limit State (ULS). Internal forces are calculated by conducting an elastic analysis for seismic forces reduced by an empirical behaviour (force-reduction) factor q representing the ability of the structural system to develop inelastic response [2]. Then, prescriptive rules are used (i.e. member detailing rules, capacity design principles) to ensure that the system is able to develop ductility capacity adequate to justify the behaviour factor employed in the calculation of internal forces. This procedure is indirect and opaque [3].

It is established that structural and non-structural damage is directly related to member deformations and lateral drifts [4, 5]. Hence, displacement- or deformation-based design represents a more rational and direct approach for controlling induced seismic damage. A number of different deformation-based seismic design methodologies (e.g. [6-8]) have been presented in the literature and an interesting comparative study of them can be found in [9].

In addition to the above, in traditional seismic design, as implemented in Eurocode 8, a single level of seismic action is examined (typically with 10% probability of exceedance in 50

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years or return period of 475 years). Only non-structural elements are checked for a more frequent seismic action at the Serviceability Limit State (SLS).

The need for improved control of structural damage for different levels of seismic action has led to the development of performance-based seismic design [10]. 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 deformation- and performance-based seismic design and assessment methodology for various levels of seismic hazard [3, 11]. MC2010 will serve as a basis for future codes for concrete structures. It is worth noting that EC8-Part 3 [12] has already adopted a performance and displacement-based methodology similar to MC2010. However, EC8-Part 3 is solely directed to seismic assessment of existing structures. MC2010 performance-based methodology covers both seismic design of new and assessment of existing structures [3].

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 structural 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 [3].

In structural engineering, the need for cost-effective design solutions of complex problems in limited time has led to the development of automated structural optimization methodologies. These can be divided in two categories: gradient-based and heuristic. Heuristic algorithms (e.g. Genetic Algorithms GA, Simulated Annealing SA, Particle Swarm Optimization PSO, Taboo Search TS) are becoming more and more popular in structural optimization, because they can handle more complicated structural problems and they don’t require calculation of derivatives [13].

Extensive research has been conducted over the past decades on optimum seismic design of structures (e.g. [14, 15]). 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 [16]. Early efforts to optimise seismic design of concrete structures were based on traditional seismic design code approaches (e.g. [17]). The number of research studies on optimization of performance- and deformation-based seismic design of reinforced concrete structures is rather limited.

Ganzerli *et al*. [18] were the first, to the best of the author’s knowledge, to consider seismic optimization with performance-based constraints. The constraints were expressed in terms of plastic rotations at column and beam members ends based on FEMA-273 guidelines (FEMA 1997). Pushover analysis was used to calculate seismic demands. Material cost was defined as the single design objective. Section dimensions and longitudinal reinforcing steel areas were set as the design variables of the optimization problem. A simple portal frame case study was examined.

Chan and Zou [19] examined optimum seismic design of reinforced concrete frames by employing optimality criteria approach. The proposed solution is divided in two steps. First, member section dimensions are selected to fulfill the serviceability performance level for frequent earthquakes. Then, member steel reinforcement is designed to withstand demands of rare earthquakes for the ultimate performance level. Pushover analysis is used to calculate seismic demands.

Lagaros and Papadrakakis [20] 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. For the latter approach, pushover analysis was employed to determine demands for different levels of earthquake intensities. Storey drifts were used as performance level indicators. Construction cost and storey drifts for the 10% probability of exceedance in 50 years (10/50) hazard level were set as the two design objectives. It was found that EC8 optimum designs are more vulnerable to future earthquakes compared to optimum designs obtained by the performance-based methodology.

Fragiadakis and Papadrakakis [21] 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. Three performance levels (Immediate Occupancy, Life Safety and Collapse Prevention) were considered. The sum of concrete and steel material costs was set as the single design objective. Design variables were determined by using tables of concrete sections and applying the concept of multi-database cascade optimization. Both, a deterministic and a reliability-based approach, were implemented. It was found that both approaches lead to structures of improved seismic resistance and reduced cost. Furthermore, reliability-based optimization may provide further economy compared to the deterministic solution.

Gencturk [22] investigated performance-based seismic design optimization of reinforced concrete and reinforced engineered cementitious composites (ECC) frames, by using Taboo Search optimization algorithm. Initial cost and seismic performance, in terms of inter-storey drifts for the 10/50 hazard level, represent the design objectives. Initial cost accounts for both material and labor costs. Performance levels are determined by inter-story drifts threshold values. These values are taken either as constant, in accordance with FEMA-273 provisions, or by mapping local strain limits to inter-story drifts after conducting sample pushover analyses. The life-cycle cost is also calculated for the optimal solutions. It is concluded that ECC can considerably improve life-cycle performance of buildings.

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 fill this gap, this study presents optimum seismic design solutions of reinforced concrete frames obtained by MC2010 and compares them with optimum designs following EC8 guidelines. To serve this goal, a general computational optimization framework of reinforced concrete frames is developed that makes use of a genetic algorithm able to track global optima of complicated problems with discrete design variables.

The aim here is to examine if and to what extent MC2010 provides more cost effective and safe design solutions with respect to EC8. This is important since MC2010 is meant to serve as a basis for future Eurocodes. In addition, topics related to the complexity and computational cost of performing seismic designs based on the two standards as well as some open issues in the seismic design provisions of MC2010 are discussed.

2 Optimization of reinforced concrete frames with genetic algorithms

2.1 Introduction

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 structural 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 (*EDPs*) (e.g. forces, displacements, rotations, drifts) or to detailing rules set by design codes and construction practice. Furthermore, 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. The previous can be written as:

Minimize: $C(\boldsymbol{x})$

Subject to: $g_j(\boldsymbol{x}) \leq 0,\ j = 1 \ to \ m$ (1)

Where:

$$\boldsymbol{x} = (x_1, x_2, \ldots, x_n)$$
$$x_i \in D_i = \left(d_{i1}, d_{i2}, \ldots, d_{ik_i}\right), i = 1 \ to \ n$$

2.2 Genetic Algorithms (GA)

Genetic Algorithms (GA) [23] belong to the class of stochastic, nature-inspired heuristic algorithms. They are based on Darwin's theory of natural selection and evolution. GA can be easily implemented and applied to advanced optimization 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 [13].

GA iteratively modify 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. The best objective function of a generation is the smallest objective function of all individuals of the generation. In order to create the next population, GA select certain individuals in the current population (parents) and use them to create individuals in the next generation (children).

The following are the basic steps of GA:

1. A random initial population is created.
2. New populations are generated successively by:
   i) Calculating the objective functions of all individuals.
   ii) Selecting parents based on their objective function.
   iii) Making children from selected parents.
   iv) Forming new population from children.
3. Algorithm is terminated when one stopping criterion is met.

Three types of children can be created by GA:

i) Elite children: These are the individuals with the best objective functions of the current population. They progress unchanged to the next population.
ii) Cross-over children: They are derived by mixing the genes of a pair of parents.
iii) Mutation children: They are created by altering the genes of a single parent.

In this study, the mixed integer GA as implemented in MATLAB-R2015a [24] is employed. This algorithm can handle both continuous and discrete design variables. To serve this goal, special crossover and mutation functions are used to ensure that discrete variables take values only from pre-determined discrete sets of values [25].

Furthermore, the algorithm is able to account for nonlinear constraints by using the penalty function approach. According to this approach, GA minimize a penalty function that is equal to the objective function plus a term accounting for constraints violation. More particularly, the penalty function is equal to the objective function for feasible designs. For an unfeasible design, however, the penalty function becomes equal to the maximum value of the objective functions of all feasible individuals of the population plus a sum of the constraint violations of the unfeasible design [26].

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 objective function value does not exceed a pre-specified tolerance over a pre-specified number of generations.

2.3 Design parameters and variables

The input data of the optimization problem can be divided in design parameters and design variables. Design parameters keep their values fixed in the optimization process. In this study, as design parameters are assumed the geometry (number and lengths of bays, story heights and member connectivity), loading and material properties of the reinforced concrete frames as well as the concrete cover of the member sections.

On the other hand, design variables determine dimensions and steel reinforcement of section properties. As shown in Fig. 1, design variables can be grouped in column and beam section properties design variable sub-vectors. Assembly of these sub-vectors forms the design variables vector $\boldsymbol{x}$.

Column section properties 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 $\boldsymbol{n_c}$ of main bars per side, assumed herein 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 herein for simplification purposes.

Beam section properties 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 $\boldsymbol{n_{tb}}$ at the top, the diameters $\boldsymbol{d_{bb}}$ and numbers $\boldsymbol{n_{bb}}$ of main bars at the bottom, the diameters $\boldsymbol{d_{bwb}}$, spacings $\boldsymbol{s_b}$ and numbers of legs $\boldsymbol{n_{wb}}$ of transverse reinforcement parallel to beam section heights.

It is important to mention here that the allocation of design variables to the section properties is independent among the sub-vectors. This means, for example, that two column section properties can have the same height and width design variables, but different number of main bars or spacing of transverse reinforcement design variables. This approach is efficient because it minimizes the use of design variables and avoids the application of equality constraints that complicates further the optimization problem.

After defining section properties, member properties need to be determined. Member properties include design parameters like member lengths, concrete cover and material properties as well as the section properties of the members. In this study, three section properties per member property are assigned. The first two section properties determine the critical regions at the ends of members and the third section property determines the internal part of the member between the two critical regions. However, the approach followed herein can be easily extended to consider an unlimited number of section properties per member property. This could be useful, for example, for beam members dominated by gravity loads, where the longitudinal and transversal reinforcement may vary significantly inside the member.

Having established member properties, groups of members having the same member properties can also be defined. This can be very effective for optimization problems since it reduces importantly the number of design variables. Furthermore, it is in agreement with typical construction practice, where several members are constructed in the same way for improving the efficiency of construction. On the other hand, this approach leads to increase of the material cost since some members are over-designed. Thus, a balance between cost and simplicity of construction is necessary.

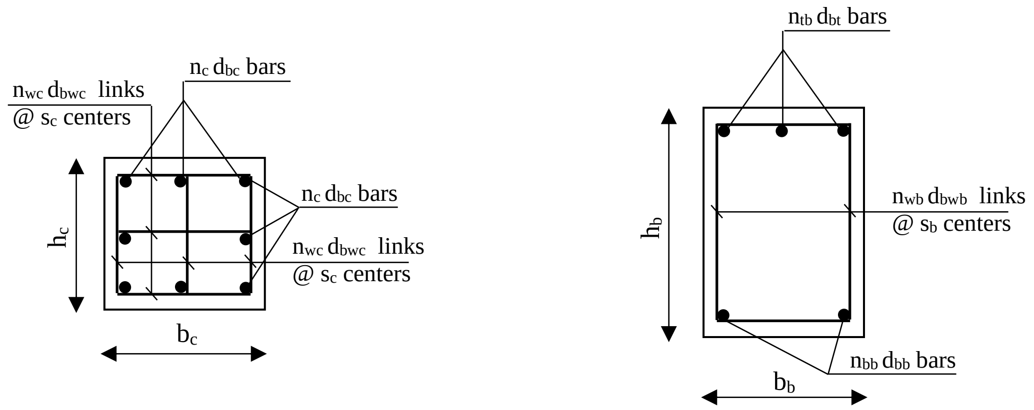

Fig. 1: Design variables: a) column sections; b) beam sections

2.4 Objective function

In this study, the objective function *C(**x**)* is the material cost of the reinforced concrete frames. The material cost consists of the cost of concrete, steel and the cost of the formworks of beam and column members. Hence, the total construction cost is taken as

$$C(x) = \sum_{i=1}^{n_{cols}} C_c^i + \sum_{i=1}^{n_{beams}} C_b^i \tag{2}$$

Where n_{cols} and n_{beams} are the numbers of column and beam members and C_c^i and C_b^i are the costs of the i^{th} column and beam member respectively. The cost of the i^{th} column and/or beam member can be determined as:

$$C_m^i = C_{cm}^i + C_{sm}^i + C_{fm}^i \tag{3}$$

Where *m* stands for column or beam (*m*=*c* or *m*=*b* respectively), C_{cm}^i is the cost of concrete, $C_{sm}^i$the cost of reinforcing steel and $C_{fm}^i$is the cost of formwork. The cost of concrete is calculated by

$$C_{cm}^i = h_m^i \cdot b_m^i \cdot L_m^i \cdot C_{cu} \tag{4}$$

Where L_m^i is the member length and C_{cu} is the cost of concrete per unit volume (Euros/m^3). Furthermore, the cost of reinforcing steel is given by:

$$C_{sm}^i = \sum_{j=1}^{3}\left[\left(A_{sj}^i \cdot L_j^i\right) + \left(V_{swj}^i \cdot L_{wj}^i / s_{wj}^i\right)\right] \cdot C_{su} \cdot \rho_s \tag{5}$$

Where A_{sj}^i and L_j^i are the area and development lengths of main reinforcing bars of section *j* of member *i*. It is noted that the lengths L_j^i of section main bars, in the case of beam members, can be different for the top and bottom main reinforcing bars. For simplicity, it is assumed in this study that the lengths $L_j^i$are equal to 25% of member length for the two end sections and 50% of member length for the internal section.

In addition, V_{swj}^{i}, L_{wj}^{i} and s_{wj}^{i} are the volume, development length and spacing of transverse reinforcement of section j of member i. V_{swj}^{i} is calculated by multiplying the total length of transverse reinforcement at section j by the area of one shear reinforcement leg. C_{su} is the cost of steel per unit mass (Euros/kg) and ρ_s is reinforcing steel density in kg/m^3. L_{wj}^{i} is taken equal to the lengths of the critical end regions for the two end sections and equal to length of the member outside the critical end regions for the internal section.

The cost of formwork of each member is determined by the following relationship, where C_{fu} is the cost of formwork per unit area (Euros/m^2).

$$C_{fm}^{i} = 2 \cdot \left(h_m^i + b_m^i\right) \cdot L_m^i \cdot C_{fu} \tag{6}$$

In the rest of this study, the following unit costs are assumed: C_{cu}=100Euros/m^3, C_{su}=1Euro/kg and C_{fu}=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 (*EDPs*) (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

$$EDP \leq EDP_{cap} \rightarrow \frac{EDP}{EDP_{cap}} - 1 \leq 0 \tag{7}$$

Regarding detailing requirements, the constraints can be expressed in terms of structural design parameters *SDPs*. 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:

$$SDP \leq SDP_{max} \rightarrow \frac{SDP}{SDP_{max}} - 1 \leq 0 \tag{8}$$

In the 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:

$$SDP \geq SDP_{min} \rightarrow \frac{SDP_{min}}{SDP} - 1 \leq 0 \tag{9}$$

In the following sections, the constraints set by the different design guidelines will be described in more detail. In addition to them, constraints related to standard construction practice should also be applied. Examples of these constraints are that the width of a beam cannot be greater than the width of the adjacent column; section dimensions of the upper parts of a column cannot be greater than section dimensions of the lower parts of the same column; number of legs of shear reinforcement cannot be greater than the number of longitudinal bars and others.

3 Optimum design of RC frames according to EC2

Prior to designing for seismic actions, RC frames must be designed to resist dead and live loads. Eurocode 2 [27] 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. (9). 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. (8).

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.

$$S_d = 1.35G_k + 1.50Q_k \tag{10}$$

EDPs constraints are written in the general form of Eq. (7), where capacities are derived by using characteristic material strengths divided by partial safety factors equal to γ_c=1.50 for concrete and γ_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 of RC frames according to EC8

In seismic design of reinforced concrete frames according to Eurocode 8 (EC8), structural members are designed to meet the Life Safety (LS) performance level for a 'rare' earthquake event with 10% probability of exceedance in 50 years (10/50) for ordinary structures. Collapse Prevention (CP) limit state is later accomplished by a number of capacity design principles. Seismic actions are defined through national zonation maps in terms of peak ground accelerations on rock a_{gR}.

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.

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. ψ_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.

$$S_{Ed} = G_k + \psi_2 \cdot Q_k + E_d \tag{11}$$

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 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 (2nd order) effects are considered at the *i* storey level with calculating ratio θ^i from Eq. (12). 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 θ^i never exceeds 0.2. Furthermore, if θ^i exceeds 0.1 then 2nd order effects are taken into account by multiplying 1st order effects by the magnification factor 1/(1-θ^i).

$$\theta^i = \frac{N_{tot}^i \cdot \Delta\delta^i}{V_{tot}^i \cdot H^i} \tag{12}$$

All previous requirements are regarded as *EDPs* constraints and are included in the optimization problem in the general form of Eq. (7). The *EDPs* are member bending moments and shear forces, interstorey drifts and θ^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. (9). 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. (8). 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.

Fig. 2a presents the flowchart of the optimization solution adopted in this study for seismic design of RC frames in accordance with EC8 provisions. It can be seen that design candidate solutions are first checked for construction practice constraints and detailing constraints according to EC2 and EC8 provisions. Detailing constraints are examined first because they require less computational effort. If detailing constraints are not satisfied then *EDPs* constraints are not checked to avoid the relatively high computational cost related to finite element analyses of RC frames. In addition, if the candidate solutions are not adequate against static loads in accordance with EC2 principles they are not examined for seismic loads to avoid unnecessary analyses.

5 Optimum seismic design of RC frames according to *fib* MC2010

fib MC2010 adopts a fully-fledged performance-based seismic design methodology [3]. 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 [3].

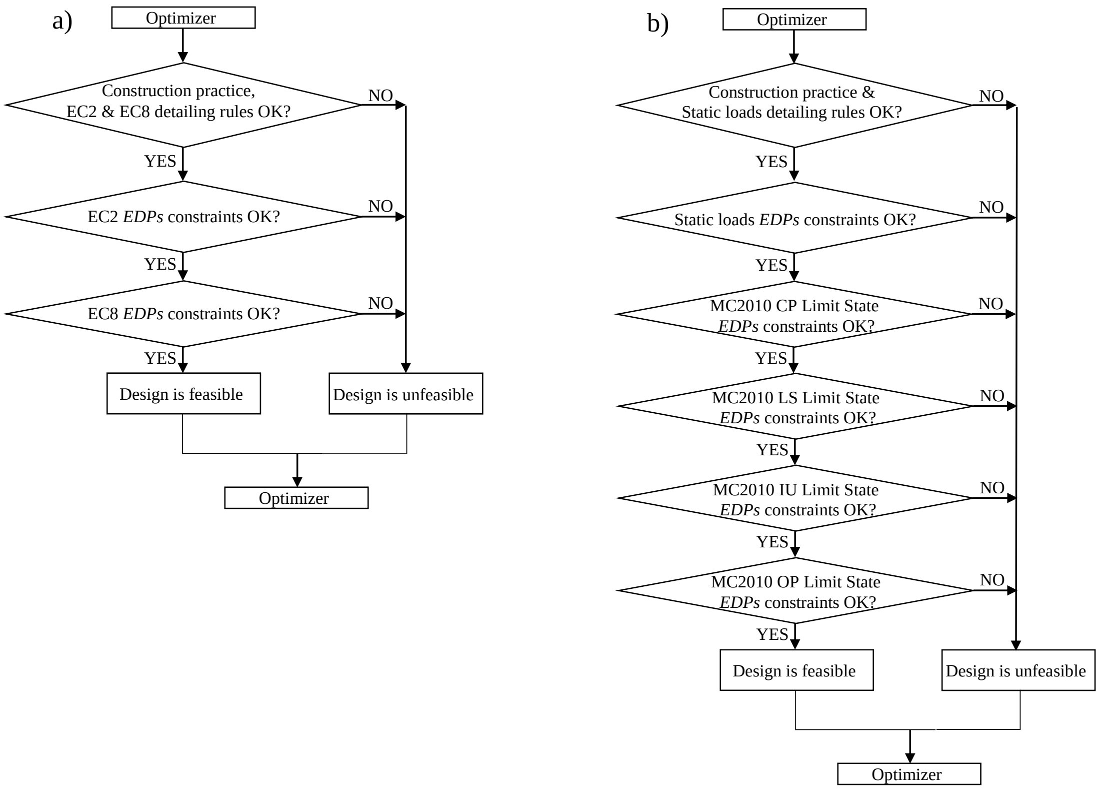

Fig. 2: Flowchart of optimum seismic design according to: a) EC8; b) MC2010

The verification of Limit States entails comparisons of chord rotation demands θ_{Ed} at member ends with yield chord rotations θ_y at the same locations for the OP Limit State and twice θ_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.

It is recommended [3] that for beams and rectangular columns with ribbed bars yield chord rotation θ_y is taken from the following equation, where φ_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.

$$\theta_y = \frac{\varphi_y(Ls + a_{scr} \cdot z)}{3} + 0.0014 \cdot \left(1 + \frac{1.5h}{L_s}\right) + \frac{\varphi_y d_{bL} f_{yl}}{8\sqrt{f_c}} \tag{13}$$

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

$$\theta_{um}^{pl} = 0.0143 \cdot 0.25^{v} \cdot f_c^{0.2} \cdot \left(\frac{\max(0.01;\omega_2)}{\max(0.01;\omega_1)}\right)^{0.3} \cdot \left(\min\left(9;\frac{L_s}{h}\right)\right)^{0.35} \cdot 25^{\left(\frac{a\rho_w f_{yw}}{f_c}\right)} \qquad (14)$$

In Eq. (14), ω_1 and ω_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. (14) 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 θ^{pl} exceeds $2\cdot\theta_y$ and 21.8° for elastic response (θ^{pl}=0). Interpolation is allowed for intermediate values of θ^{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.

$$EI_{eff} = \frac{M_y L_s}{3\theta_y} \qquad (15)$$

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 $2\cdot T$, 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. (15). Hence, a post-design check is required to verify that the set of ground motions satisfies the selection criteria of MC2010 based on the actual T of the design solution.

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

Limit State	Seismic Hazard	Deformation Limit
Operational (OP)	Frequent with 70% probability of exceedance in 50 years (70/50)	Mean value of θ_y
Immediate Use (IU)	Occasional with 40% probability of exceedance in 50 years (40/50)	Mean value of θ_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 γ^*_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 (γ^*_R =1)

Fig. 2b presents the optimum design methodology adopted in this study for seismic design of RC frames in accordance with MC2010. Initially, the design solutions are examined for construction and static loads detailing rules and *EDPs*. This is done in a manner similar to optimum design according to EC2. These constraints are checked first because they require significantly less computational effort than the time consuming nonlinear response history analyses. Later, the *EDPs* are examined successively for each Limit State of MC2010. If one Limit State is not satisfied then the following ones are not examined to avoid unnecessary response history analyses. All *EDPs* constraints are written in the general form of Eq. (7).

Even with this approach, it is clear that MC2010 requires a large number of inelastic response history analyses to be conducted for each design solution. This increases grossly the computational cost of the optimization task, where a significant number of trial designs need to be examined in order to obtain the optimum solution.

Before closing this section, it is mentioned that no specifications of the MC2010 are provided regarding serviceability checks of non-structural components as well as some detailing rules concerning for example the length of the critical regions, where enhanced ductility demands are expected. To fill this gap in this study, serviceability checks of non-structural components are conducted according to EC8 recommendations and critical end region lengths are calculated in accordance with EC8 DCM specifications.

6 Optimum seismic design of RC frames applications

In this section, applications of the optimum seismic design methodologies described previously to RC plane frames are presented. In particular, a simple portal frame and a concrete frame with 4 storeys and 2 bays are examined. The buildings are of ordinary importance and rest on soil class B according to the classification of EC8. The frames are 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 (low, moderate and high respectively) on the optimum seismic design solutions. The elastic (target) response spectrum with 5% damping of EC8 determined for the previous specifications and 0.24g peak ground acceleration is shown in Fig. 3.

Peak ground accelerations for the other seismic hazard levels of MC2010 objectives are calculated by multiplying the 10/50 values by the importance factor γ_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%).

$$\gamma_I = \left(\frac{P_L}{P_{LR}}\right)^{-1/3} \quad (15)$$

The frames are 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 specifications, 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 MC2010 specifications. The goal here is to examine to which extent the conservative specifications of MC2010 on the selection of ground motion records, described in section 5, can influence the cost of the optimal design solutions with respect to EC8 ground motion selection provisions.

Figure 3a presents the scaled and mean elastic spectra with 5% damping of the set of 7 ground motions selected and scaled following EC8 provisions. In this case, selection and scaling was performed by employing computer program REXEL [28]. 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 can be seen in Table 2. They are all recorded on soil type B and have magnitude M_w>5.5. It is evident in Fig. 3a that the mean spectrum follows very closely the target spectrum.

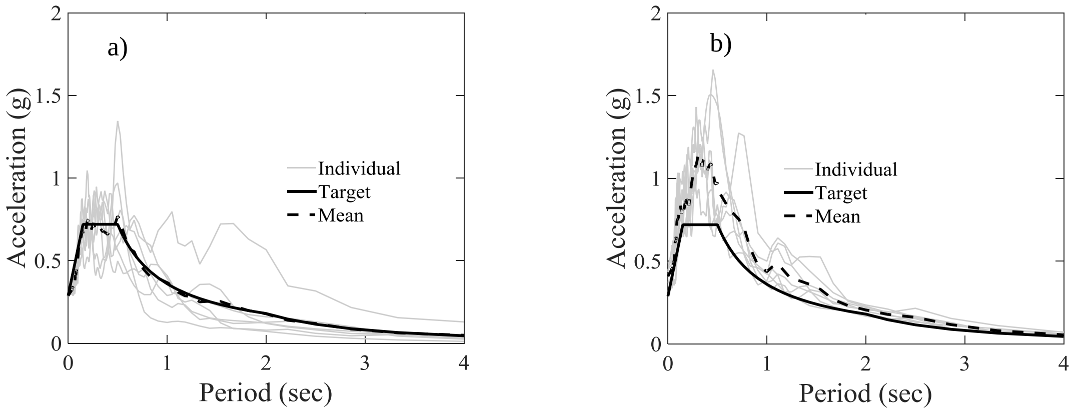

Fig. 3: Elastic spectra with 5% damping for ground motion sets selected and scaled in accordance with a) EC8; b) MC2010

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 [29] 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 ranging 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 [30]. The first 7 ground motions comprise the set of records used herein (Table 2). Figure 3b 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. This reflects the level of conservatism adopted in MC2010 specifications.

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; … ; 1.5m). 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.

Table 2: Unscaled ground motions selected based on EC8 and MC2010 provisions

Records selected based on EC8						
Earthquake Name	Station	Year	Epicentral Distance R (km)	Magnitude M_w	PGA (g)	Direction
Kalamata	ST163	1986	11	5.9	0.24	X
Montenegro (aftershock)	ST77	1979	20	6.2	0.06	Y
Izmit	ST859	1999	73	7.6	0.12	Y
South Iceland	ST2484	2000	7	6.5	0.51	Y
Umbria Marche	ST83	1997	23	6	0.08	X
Friuli (aftershock)	ST24	1976	14	6	0.34	Y
Aigion	ST1330	1995	43	6.5	0.03	Y
Records selected based on MC2010						
Earthquake Name	Station	Year	Epicentral Distance R (km)	Magnitude M_w	PGA (g)	Direction
Kalamata	ST163	1986	10	5.9	0.24	X
Kalamata	ST164	1986	11	5.9	0.21	X
South Iceland	ST2484	2000	7	6.5	0.51	Y
Campano Lucano	ST99	1980	33	6.9	0.10	X
South Iceland	ST2482	2000	15	6.5	0.48	Y
Ano Losia	ST1257	1999	18	6	0.09	Y
Friuli	ST14	1976	42	6.5	0.09	Y

6.1 Portal frame

In this section, a simple portal reinforced concrete frame (Fig. 4a) is optimally designed in accordance with the methodologies described previously. 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 joints equal to 120.0kN for permanent and 80.0kN for 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, B1 has the same top and bottom longitudinal reinforcement and member end sections have the same transverse reinforcement. Furthermore, due to construction reasons, it is assumed that the longitudinal reinforcement does not vary along beam and column members. However, end and intermediate sections may have different transverse reinforcement spacing to account for the additional design requirements in the critical end regions.

Following these observations, two column and two beam sections are used as shown in Fig. 4a. Sections 1 are used for member end zones and sections 2 for the rest of the element. Sections 1 and 2 have exactly the same detailing apart from spacing of transverse reinforcement. In total, 16 (8 for columns and 8 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-independent GA runs for each design solution were conducted and the minimum cost obtained is reported herein.

Figure 4b 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 optimum cost increases as design accelerations increase.

Figure 5a compares optimum costs in Euros obtained by all seismic design methodologies for the three design peak ground accelerations for 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 hand, 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. It is also worth noting that the optimum costs of DCH remain essentially the same for all design PGA values. This shows the influence of detailing requirements on the final costs of reinforced concrete structures.

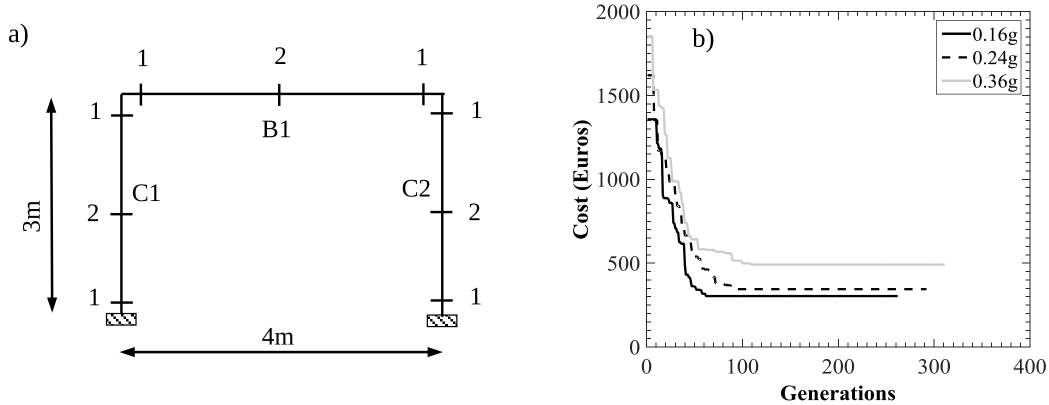

Fig. 4: a) Examined portal frame; b) Optimization histories of designs obtained by MC2010 methodology for THA ground motion set and three different design PGAs

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. Furthermore, the MC2010 design with THA motion set yields slightly smaller cost than the EC8 designs 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. 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 higher seismicity levels the cost derived by selecting a ground motion data set in accordance with MC2010 provisions is significantly higher than the one derived by the EC8-Part 1 compatible set of accelerograms.

Figure 5b shows percentile contributions of construction cost components to the total cost obtained by the different design methodologies for all design PGAs. It can be seen that for the 0.16g designs according to MC2010 concrete and formwork dominate structural cost. However, this changes as design PGA increases and for the 0.36g design for THB motion set the steel contributes more to the total cost. It is also worth noting the increased contribution of transverse steel for the DCH design with respect to the other two EC8 ductility classes.

Table 3 presents section dimensions, longitudinal reinforcement ratio ρ_l and ratio of transverse steel parallel with the shear force ρ_w of the optimum solutions. It can be seen in this table that the THA design solutions have always smaller ρ_l values than the DCL solutions (for similar section sizes) and smaller, similar or even larger ρ_l values than the DCM solutions. Furthermore, they have equal or larger ρ_w values than the DCL solutions and smaller ρ_w values than the DCM solutions. It is also worth noting that the THA solutions have the same transverse reinforcement ratios inside and outside the critical end zones. This is the case because the provided transverse reinforcement is adequate to satisfy the rotation and shear force constraints at the member ends and no additional detailing and confinement requirements are set by MC2010 inside the critical end regions.

Figure 6 presents MC2010 checks of rotation and shear force constraints (Eq. 7) for all Limit States as obtained by subjecting all 0.36g PGA optimum design solutions to the THA ground motion set. Column sections 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 stated by the acronyms shown in Table 1.

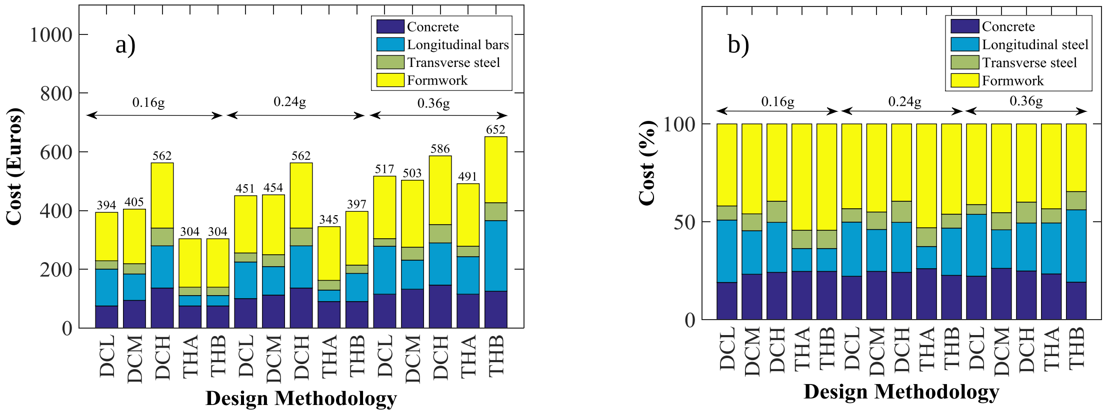

Fig. 5: Optimum costs obtained by different design methodologies and design PGAs a) in Euros; b) percentile contributions

Table 3: Section properties of optimum design solutions

Members		Columns								Beams							
Sections		Section 1				Section 2				Section 1				Section 2			
Property		h_c	b_c	ρ_l	ρ_w	h_c	b_c	ρ_l	ρ_w	h_b	b_b	ρ_l	ρ_w	h_b	b_b	ρ_l	ρ_w
Units		m	m	%	%	m	m	%	%	m	m	%	%	m	m	%	%
0.16g	DCL	0.3	0.25	2.41	0.40	0.3	0.25	2.41	0.40	0.3	0.25	0.80	0.27	0.3	0.25	0.80	0.27
	DCM	0.3	0.3	1.79	0.50	0.3	0.3	1.79	0.33	0.4	0.25	0.23	0.40	0.4	0.25	0.23	0.16
	DCH	0.4	0.4	1.57	0.59	0.4	0.4	1.57	0.29	0.4	0.25	0.40	0.40	0.4	0.25	0.40	0.16
	THA	0.3	0.25	0.60	0.40	0.3	0.25	0.60	0.40	0.3	0.25	0.30	0.27	0.3	0.25	0.30	0.27
	THB	0.3	0.25	0.60	0.40	0.3	0.25	0.60	0.40	0.3	0.25	0.30	0.27	0.3	0.25	0.30	0.27
0.24g	DCL	0.4	0.25	1.81	0.40	0.4	0.25	1.81	0.40	0.4	0.25	0.63	0.16	0.4	0.25	0.63	0.16
	DCM	0.4	0.3	1.34	0.50	0.4	0.3	1.34	0.33	0.4	0.25	0.34	0.40	0.4	0.25	0.34	0.16
	DCH	0.4	0.4	1.57	0.59	0.4	0.4	1.57	0.29	0.4	0.25	0.40	0.40	0.4	0.25	0.40	0.16
	THA	0.4	0.25	0.45	0.40	0.4	0.25	0.45	0.40	0.3	0.25	0.30	0.27	0.3	0.25	0.30	0.27
	THB	0.4	0.25	1.26	0.40	0.4	0.25	1.26	0.27	0.3	0.25	0.80	0.27	0.3	0.25	0.80	0.27
0.36g	DCL	0.5	0.25	1.93	0.27	0.5	0.25	1.93	0.27	0.4	0.25	0.79	0.16	0.4	0.25	0.79	0.16
	DCM	0.4	0.3	1.34	0.50	0.4	0.3	1.34	0.33	0.6	0.25	0.27	0.40	0.6	0.25	0.27	0.13
	DCH	0.4	0.4	1.57	0.59	0.4	0.4	1.57	0.29	0.5	0.25	0.32	0.40	0.5	0.25	0.32	0.13
	THA	0.5	0.25	1.81	0.40	0.5	0.25	1.81	0.40	0.4	0.25	0.34	0.16	0.4	0.25	0.34	0.16
	THB	0.5	0.25	3.01	0.80	0.5	0.25	3.01	0.54	0.5	0.25	0.80	0.30	0.5	0.25	0.80	0.30

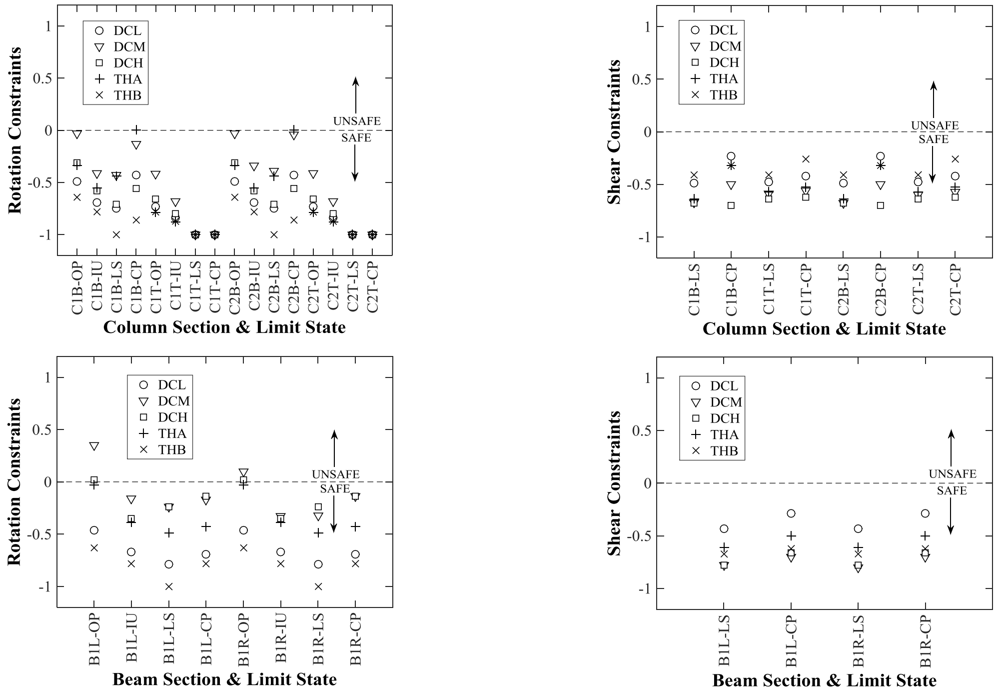

Fig. 6: MC2010 rotation and shear force constraints of beam and column section optimum solutions obtained by different design methodologies for 0.36g design PGA.

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.

6.2 Four-storey two-bay frame

In this section, a four-storey two-bay reinforced concrete frame (Fig. 7) is optimally designed according to EC8 and MC2010 provisions. Span length is 3m and storey height is 3m. Concrete C25/30 and reinforcing steel B500C are used. Concrete cover is assumed to be 30mm. Vertical concentrated loads of 144.0kN are applied at all exterior joints and 288kN at the interior joints. All storey masses for the seismic load combination are equal to 59.9t.

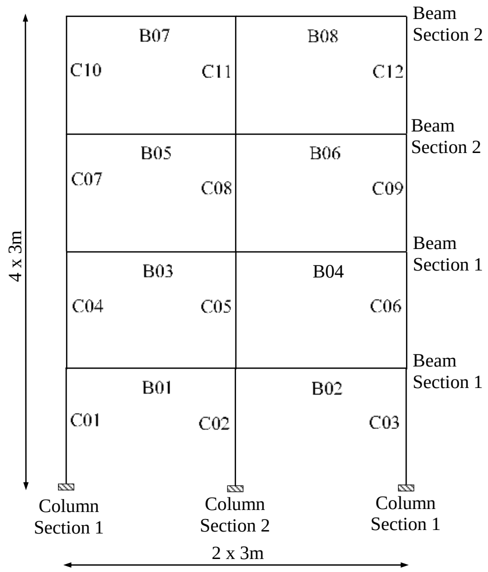

Fig. 7: Examined four-storey two-bay frame

The frame consists of 12 columns and 8 beams. Due to symmetry, it is assumed that the two exterior columns have exactly the same sections and reinforcement. Furthermore, for the simplicity of the calculations, it is assumed that sections and reinforcement remain the same along columns height. It is also assumed that one bar diameter is used for all longitudinal reinforcement bars of the exteriors and interior column. The same holds for the diameter of transverse reinforcement placed in all columns.

Regarding beam members, it is assumed that the beams of the 1^{st} and 2^{nd} storey have the same section and steel reinforcement, which is uniform along their length. The same assumption is made for the beams of the 3^{rd} and 4^{th} storey. It is also assumed that one bar diameter is used for all beam longitudinal reinforcing bars and one bar diameter for the transverse reinforcement of all beam members. Due to symmetry, it is also assumed that beam sections have the same top and bottom longitudinal reinforcement.

Following the previous observations, two different column section properties and two different beam section properties are used in this study. Column section 1 is used for the exterior and column section 2 for the interior columns. Beam section 1 is used for the beams of the first 2 storeys and beam section 2 for the beams of the last two storeys. In total, 23 independent design variables are employed for this problem.

The results presented in the following were obtained by running GA with populations of 100 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-independent GA runs for each design solution were conducted and the minimum cost obtained is reported herein.

Figure 8a compares optimum costs in Euros obtained by all seismic design methodologies for the three design peak ground accelerations for the 10/50 seismic hazard level. It can be seen that in all cases costs increase as design accelerations increase. Among EC8 design solutions, DCM yields the most cost effective solution for all seismicity levels. The cost of DCL is less than the cost of DCH for 0.16g design PGA, but increases sharply for higher levels of PGAs. Hence, DCL becomes the most expensive EC8 solution for 0.24g and 0.36g PGAs.

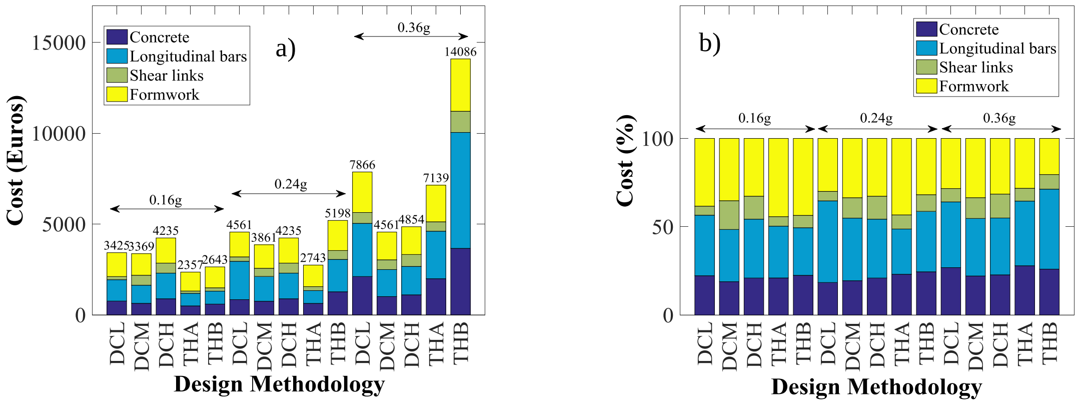

Fig. 8: Optimum costs obtained by different design methodologies and design PGAs a) in Euros; b) percentile contributions

It can also be seen 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 PGA. For the moderate 0.24g design PGA, THA motion set yields significantly reduced cost, but THB motion set drives to more expensive solutions than EC8. The MC2010 design with THA motion set yields similar cost to DCL for 0.36g design PGA and significantly higher cost than DCM and DCH. The same design methodology with the THB motion set drives to importantly higher costs (1.8-3.1 times) than all EC8 designs obtained for 0.36g design PGA.

Similarly to the portal frame, the comparison of optimum costs obtained by MC2010 methodology for the THA and THB ground motion sets shows the importance of the ground motions set. For 0.16g PGA both solutions yield similar costs. This is because the design in this case is controlled by minimum detailing requirements. However, for larger seismicity levels the cost derived by selecting a ground motion data set in accordance with MC2010 provisions is significantly higher than the one derived by the EC8-Part 1 set ground motions (90% and 97% for 0.24g and 0.36g design PGA respectively). Furthermore, Fig. 8b shows percentile contributions of construction cost components to the total cost obtained by the different design methodologies for all design PGAs. The same conclusions as the ones derived for the portal frame (Fig. 5b) hold for the frame examined in this section.

Table 4 presents section dimensions and reinforcement ratios ρ_l and ρ_w of the optimum solutions. Due to complexity, it is difficult to compare the optimum solutions and derive safe conclusions. However, it can generally be observed that for the low and moderate seismicity the THA solution yields the smallest beam and column section sizes. For the high seismicity level, THA solution has similar column sizes to DCL and similar beam sizes to DCH. It is also noted that THA yields typically small ρ_w values that are generally close to DCL and sometimes smaller for similar section sizes. The latter is attributed to the fact that DCL transverse reinforcement design is dominated by high shear force demands in these cases.

Figure 9 presents MC2010 checks of rotation and shear force constraints (Eq. 7) for the beams and column member of the first 2 storeys and for all Limit States as obtained by subjecting all 0.36g PGA design solutions to the THA ground motion set. The same designations are used as in Fig. 6. It can be seen that EC8 solutions fail to satisfy a considerable number of beam rotation constraints set by MC2010 and in one case (DCM) a column rotation constraint. Regarding shear constraints, DCM and DCH provide safe designs due to shear capacity design principles. However, DCL design solution violates in several cases shear force constraints especially in the case of beam members. Regarding MC2010 design for the THA ground motion set, it can be seen that the rotation constraints of beam members are in many cases close to zero (still on the safe side), which means that they dominated this design solution. MC2010 design for the THB motion set provides generally conservative results.

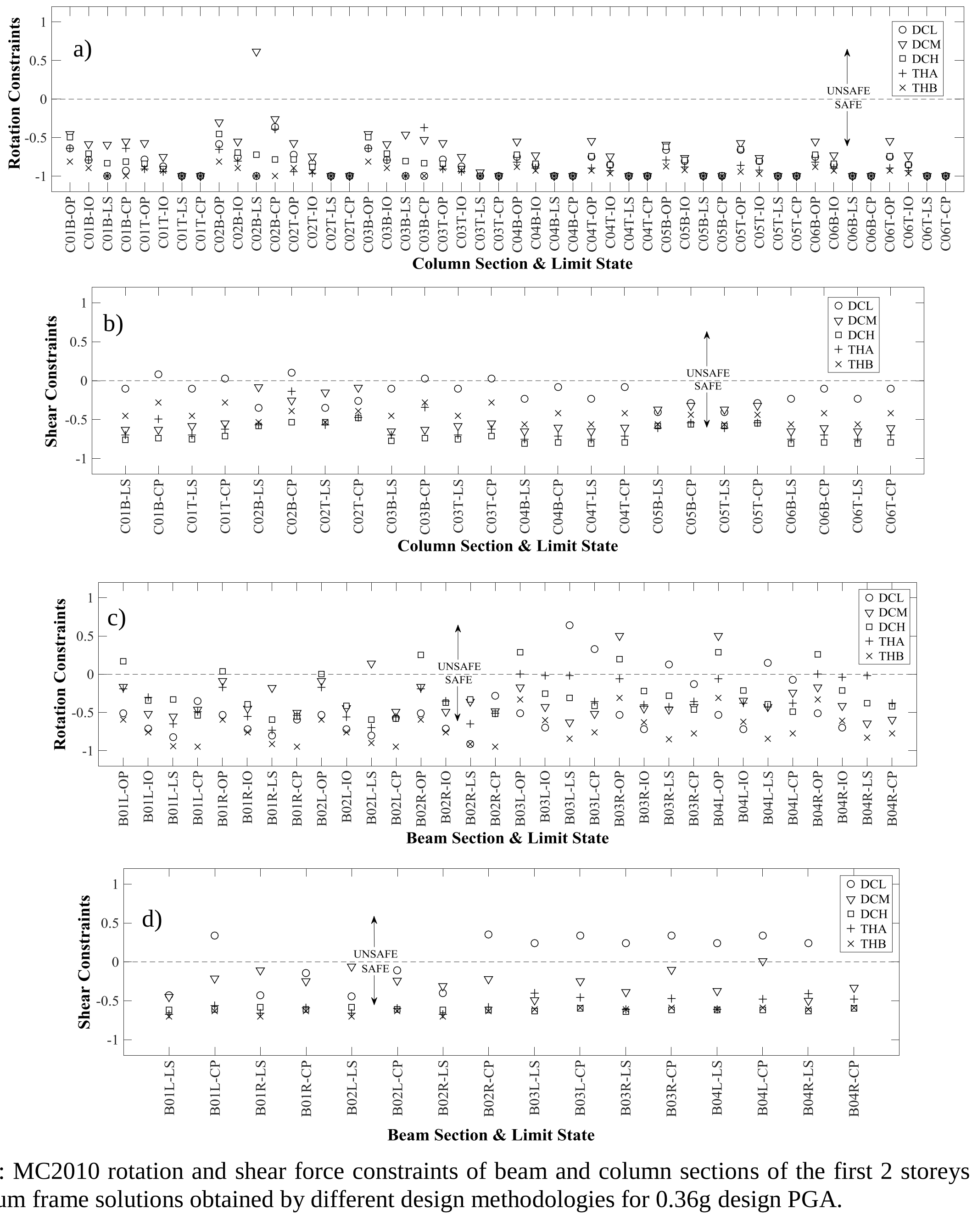

Fig. 9: MC2010 rotation and shear force constraints of beam and column sections of the first 2 storeys of the optimum frame solutions obtained by different design methodologies for 0.36g design PGA.

Furthermore, Table 5 provides information regarding the fundamental period T and maximum base shear V_{max} of all the 0.36g design PGA optimum solutions. It can be seen that the maximum V_{max} and minimum T are developed for the DCL design and the opposite happens for the DCH solution. It is noted that, for direct comparison reasons, all periods in Table 5 are determined by using MC2010 specifications according to which member stiffness is proportional to member moment capacity. As a result, the DCL solution that is designed for higher moment demands (lower q factor) has significantly lower T than DCH. Regarding MC2010 solutions, it is observed that the period and base shear of the THA solution lie between the DCL and DCH limits and they are very close to the DCM solution. The THB solution has similar period to the THA, but significantly higher base shear.

Table 4: Section properties of optimum design solutions

Members		**Columns**								**Beams**							
Sections		**Section 1**				**Section 2**				**Section 1**				**Section 2**			
Property		h_c	b_c	ρ_l	ρ_w	h_c	b_c	ρ_l	ρ_w	h_b	b_b	ρ_l	ρ_w	h_b	b_b	ρ_l	ρ_w
Units		m	m	%	%	m	m	%	%	m	m	%	%	m	m	%	%
0.16g	**DCL**	0.4	0.3	2.01	0.22	0.4	0.3	2.68	0.22	0.5	0.3	0.94	0.22	0.5	0.25	0.64	0.20
	DCM	0.3	0.3	2.68	0.67	0.5	0.3	2.14	0.84	0.4	0.25	0.80	0.90	0.4	0.25	0.45	0.90
	DCH	0.4	0.4	2.36	0.50	0.5	0.4	2.51	0.63	0.4	0.3	0.67	0.50	0.4	0.25	0.40	0.40
	THA	0.3	0.25	2.14	0.27	0.4	0.25	3.22	0.27	0.4	0.25	0.23	0.16	0.25	0.25	0.36	0.27
	THB	0.4	0.25	1.61	0.40	0.4	0.3	2.68	0.22	0.3	0.25	0.30	0.27	0.4	0.25	0.34	0.24
0.24g	**DCL**	0.5	0.3	3.22	0.22	0.4	0.4	3.01	0.17	0.4	0.3	1.84	0.42	0.4	0.3	1.31	0.52
	DCM	0.4	0.3	3.14	0.67	0.4	0.4	2.36	0.50	0.5	0.25	0.80	0.40	0.4	0.25	0.60	0.40
	DCH	0.4	0.4	2.36	0.50	0.5	0.4	2.51	0.63	0.4	0.3	0.67	0.50	0.4	0.25	0.40	0.40
	THA	0.4	0.3	1.13	0.33	0.5	0.25	2.17	0.40	0.4	0.25	0.57	0.16	0.25	0.25	0.72	0.40
	THB	0.5	0.5	1.29	0.20	0.5	0.4	2.81	0.67	0.5	0.4	0.90	0.42	0.4	0.4	1.00	0.25
0.36g	**DCL**	0.7	0.5	2.15	0.15	0.9	0.5	1.67	0.35	1	0.4	0.60	0.39	0.7	0.3	0.86	0.35
	DCM	0.4	0.4	2.01	0.50	0.6	0.3	2.68	0.67	0.8	0.3	0.47	0.33	0.4	0.25	1.13	0.40
	DCH	0.5	0.5	1.51	0.40	0.5	0.4	2.51	0.63	0.4	0.3	1.00	0.67	0.4	0.25	0.80	0.60
	THA	0.8	0.6	1.34	0.17	0.7	0.6	2.11	0.17	0.4	0.3	0.84	0.45	0.4	0.4	1.26	0.38
	THB	0.9	0.6	2.18	0.28	1	0.8	2.21	0.28	1.2	0.6	0.89	0.28	0.9	0.5	1.53	0.27

Table 5: Response characteristics of optimum design solutions obtained by different design methodologies for 0.36g design PGA.

Design Methodology	V_{max} (kN)	T (sec)
DCL	1950.0	0.57
DCM	936.4	1.08
DCH	721.0	1.40
THA	1007.0	1.06
THB	1444.0	1.01

7 Conclusions

Eurocode 8 adopts a force-based seismic design methodology examining a single level of seismic action. In order to provide enhanced control of structural damage for different levels of seismic action, the new *fib* Model Code 2010 (MC2010) includes a fully-fledged displacement- and performance-based seismic design methodology at the expense of higher complexity and computational effort. Due to this complexity, automated optimization techniques represent an efficient and in some cases necessary tool for obtaining cost-effective design solutions.

This study presents optimum seismic design solutions of reinforced concrete frames obtained by MC2010 design procedure and compares them with optimum designs based on EC8 for all ductility classes. This is important since MC2010 is meant to serve as a basis for future Eurocodes. To serve this goal, a general computational optimization framework of reinforced concrete frames is developed that makes use of a genetic algorithm able to track global optima of complex problems with discrete design variables.

Comparisons of the costs of EC8 optimum solutions show that DCL is cost effective for low seismic demands (PGA=0.16g), but DCM and DCH become more cost effective for moderate (PGA=0.24g) and high (PGA=0.36g) seismicity levels.

Regarding MC2010, it is shown that MC2010 optimum designs are less expensive than EC8 for low design PGA values, they have similar costs to EC8 solutions for moderate design PGA values and they can get more expensive for high design PGAs.

Examination of the rotation and shear force constraints set by MC2010 for all design methodologies shows that in several cases the EC8 designs do not satisfy MC2010 constraints. This observation shows that MC2010 provides better control of structural damage than EC8. In this regard, the additional cost required in some cases by MC2010 for high seismicity designs is justified by the fact that it provides enhanced control of structural damage. On the other hand, the additional cost of EC8 solutions for low design PGAs can be seen as unnecessary. Hence, it can be concluded that MC2010 ensures a more rational allocation of structural costs.

Furthermore, it is observed that the optimum costs of MC2010 depend significantly on the specifications applied for the selection of the ground motion set. If the set of accelerograms is selected based on MC2010 conservative provisions, then optimum costs in high seismicity regions can increase by more than 100% with respect to selecting ground motions according to EC8 guidelines. It is also shown that the selection of ground motions according to MC2010 drives to over-conservative results in terms of *EDPs*. Therefore, the MC2010 specifications for the selection of ground motions could be re-examined as they may undermine the ability of the code to produce more cost-effective and sustainable structural solutions than EC8.

It is important to note that design according to MC2010 provisions involves high computational effort. This is the case because a great number of time consuming nonlinear response history analyses need to be conducted in order to verify that the design objectives are met. This issue becomes more important in the framework of structural optimisation, where a significant number of trial design solutions need to be examined in order to obtain the optimum solution. Therefore, the use of alternative analysis methods like linear response history analysis [3] or pushover analysis for the cases that they can provide reliable results needs to be further explored.

Before closing, some open issues in the specifications of MC2010 are noted like the checks of non-structural components and some detailing rules concerning the length of the critical regions, where enhanced ductility capacity is required. These could be addressed in future versions of the code.

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