AUTOMATED OPTIMUM SEISMIC DESIGN OF REINFORCED CONCRETE FRAMES WITH NONLINEAR RESPONSE-HISTORY ANALYSIS

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ABSTRACT

Performance and deformation-based seismic design represents the most rational approach to control structural damage for different levels of seismic hazard. A prerequisite of the successful application of performance-based seismic design is the accurate prediction of inelastic seismic demands that can be achieved by nonlinear response-history analysis. To this point, only a limited number of research studies has been dedicated to the optimum performance-based seismic design of reinforced concrete structures using nonlinear response-history analysis. This can be mainly attributed to the high levels of complexity and computational demand involved. The present study develops optimum performance-based seismic designs of reinforced concrete frames and compares them with optimum designs obtained by prescriptive seismic design methodologies. In addition, a novel, computationally efficient, solution strategy is proposed for the optimization problem of this study that is found to outperform the standard solution approaches.

Keywords: Structural optimization; Reinforced concrete; Seismic design; Performance-based

1. INTRODUCTION

It is established that structural and non-structural damage is directly related to member deformations and lateral drifts (Priestley et al. 2007, Fardis 2009). Hence, displacement- or deformation-based design represents a more rational and direct approach for controlling induced seismic damage. Furthermore, the requirement for direct control of structural damage for different levels of seismic action has led to the development of performance-based seismic design (Fardis 2013). Performance-based seismic design 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.

Nowadays, performance and deformation-based seismic design is emerging in seismic design standards. For example, the fib Model Code 2010 (MC2010), meant to serve as a basis for future codes for concrete structures, includes a fully-fledged deformation- and performance-based seismic design and assessment methodology for various levels of seismic hazard (Fardis 2013). In MC2010, each performance limit state is expressed in terms of deformation limits of the structural members providing direct control of allowable structural damage.

A crucial issue in the successful application of performance-based seismic design is the reliable prediction of seismic demands. In MC2010, 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.

Structural design solutions shouldn’t be only reliable but also sustainable and cost-effective (Mergos 2017b, 2018). The urgent need for cost-effective designs of complex problems in limited time has led to the development of structural optimization methodologies. These make use of automated optimization
algorithms that 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 increasingly popular in structural optimization, because they can handle more complicated structural problems and they don’t require calculation of derivatives (Yang 2014).

Extensive research has been conducted over the past decades on optimum seismic design of structures (Lagaros 2014). However, only a limited number of research studies has been dedicated to the optimum performance-based seismic design of reinforced concrete structures (e.g., Ganzerli et al. 2000, Chan and Zou 2004, Lagaros and Papadrakakis 2007, Fragiadakis and Papadrakakis 2008, Lagaros and Fragiadakis 2011, Genceturk 2013, Mergos 2017a). An even smaller number of these studies is using nonlinear response-history analysis to calculate seismic demands (e.g., Fragiadakis and Papadrakakis 2008, Mergos 2017a). In one of these studies, Mergos (2017a) compared optimum seismic designs of reinforced concrete frames designed according to the deformation and performance-based design methodology of fib Model Code 2010 and the prescriptive approach of Eurocode 8 – Part 1 (CEN 2004). It is found that the fib Model Code approach drives to significant cost savings, especially in regions of low seismicity, and controls better the levels of seismic damage.

The rather limited application of automated structural optimization techniques to the performance- and deformation-based seismic design with the aid of nonlinear response-history analysis can be mainly attributed to two reasons. First, the computational demand of nonlinear response history analysis combined with the large number of analyses required to examine reliably all performance objectives for a single design and the high number of trial designs required by the optimization algorithms drives to high computational costs. Second, nonlinear response-history analysis of reinforced concrete frames requires the knowledge of reinforcing steel bars that can increase significantly the number of design variables expanding exponentially the search space.

The main goal of this study is to present the main framework for the automated optimum performance-based seismic design of reinforced concrete frames with nonlinear response history analysis and illustrate some important findings from its application to specific case studies. Furthermore, a new computationally efficient methodology is introduced that can reduce significantly the required number of design variables of this optimization problem leading to more robust and cost-effective solutions.

2. AUTOMATED OPTIMUM SEISMIC DESIGN METHODOLOGY

2.1 Optimization problem formulation

Optimum seismic design of reinforced concrete frames can be formulated as a standard optimization problem with discrete design variables. In the case of a single objective, this optimization problem is generally written as:

Minimize: \( F(\mathbf{x}) \)
Subject to: \( g_j(\mathbf{x}) \leq 0, \ j = 1 \text{ to } m \) \( \) (1)
Where:
\[
\mathbf{x} = (x_1, x_2, ..., x_n) \quad x_i \in D_i = (d_{i1}, d_{i2}, ..., d_{iki}), i = 1 \text{ to } n
\]

In Equation 1, \( F(\mathbf{x}) \) represents the objective function to be minimized by the optimization problem. The vector \( \mathbf{x} \) is the design solution and contains \( n \) number of independent design variables \( x_i \) \( (i = 1 \text{ to } n) \). Design variables \( x_i \) take values from discrete sets of values \( D_i = (d_{ij}, d_{i2}, ..., d_{i(k_i)}), \) where \( d_{ip} (p = 1 \text{ to } k_i) \) is the \( p \)-th possible discrete value of design variable \( x_i \) and \( k_i \) is the number of possible discrete values of \( x_i \). Furthermore, the solution should be subject to \( m \) number of constraints \( g_j(\mathbf{x}) \leq 0 (j = 1 \text{ to } m) \).

2.2 Objective function

Different objective functions \( F(\mathbf{x}) \) can be considered in the optimization problem of Equation 1 (Mergos 2018). The most typical objective function addressed is the material cost of reinforced concrete frames. This cost can be taken as the sum of costs of concrete \( C_c(\mathbf{x}) \), steel \( C_s(\mathbf{x}) \) and formworks \( C_f(\mathbf{x}) \). In the rest
of this study, the material cost is used as design objective assuming the following unit prices: \( C_{co} = 100 \) Euros/m\(^3\), \( C_{so} = 1 \) Euro/kg and \( C_{fo} = 15 \) Euros/m\(^2\) for the cost of concrete, reinforcing steel and formwork respectively.

2.3 Design variables

The input data in optimization problems are divided in design parameters that keep their values fixed during the optimization process and design variables. In the sizing optimization problem examined herein, design parameters are assumed the geometry, material properties, concrete cover and external loading of the reinforced concrete frames. On the other hand, design variables determine dimensions and steel reinforcement of section properties.

The design variables vector \( \mathbf{x} \) in sizing optimization of reinforced concrete frames can be divided in three distinct design sub-vectors. The cross-sectional dimensions sub-vector \( \mathbf{x}_{cd} \), the longitudinal steel reinforcement bars sub-vector \( \mathbf{x}_{sl} \) and the transverse steel reinforcement bars sub-vector \( \mathbf{x}_{sw} \). These three sub-vectors can be later combined to form the design vector \( \mathbf{x} = [\mathbf{x}_{cd}, \mathbf{x}_{sl}, \mathbf{x}_{sw}] \). A detailed description of the design variables included in the \( \mathbf{x}_{cd}, \mathbf{x}_{sl} \) and \( \mathbf{x}_{sw} \) design sub-vectors in the simple case of rectangular beam and column sections can be found in Mergos (2017a).

2.4 Design constraints

The design constraints of the optimization problem examined herein are set by the performance-based seismic design methodology of fib Model Code 2010 (MC2010). Additionally, construction practice constraints and constraints referring to structural design for static loads must be addressed. In this study, design constraints for static loads are following the provisions of Eurocode 2 (CEN 2000). A detailed description of these constraints can be found in Mergos (2017a).

fib MC2010 adopts a fully-fledged, performance-based seismic design methodology (Fardis 2013). The code requires the verification of 4 Limit States. The Operational (OP) and Immediate Use (IU) which are serviceability Limit States (SLS) and the Life Safety (LS) and Collapse Prevention (CP) which represent Ultimate Limit States (ULS). All Limit States are verified by comparing chord rotation demands \( \theta_{Ed} \), for different levels of Seismic Hazard, with chord rotation limit values \( \theta_{lim} \) as presented in Table 1. In Table 1, \( \theta_{y} \) is the yield chord rotation of concrete members and \( \theta_{pl}^{u,k} \) is the characteristic ultimate plastic hinge rotation capacity. Furthermore, for the ULS, brittle shear failures should be prevented by ensuring that shear force demands \( V_{Ed} \) are lesser than design shear force capacities \( V_{lim} \).

In all cases, MC2010 performance constraints can be expressed in the generic form \( g_j(x) \leq 0 \) by Equation 2, where the Engineering Demand Parameters (EDPs) represent both chord rotations and shear forces.

\[
EDP \leq EDP_{lim} \Rightarrow g_j(x) = \frac{EDP}{EDP_{lim}} - 1 \leq 0 \ (j = 1 \text{ to } m) \tag{2}
\]

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Seismic Hazard</th>
<th>Deformation Limits ( \theta_{lim} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Operational (OP)</td>
<td>Frequent with 70% probability of exceedance in 50 years (70/50)</td>
<td>Mean value of ( \theta_{y} )</td>
</tr>
<tr>
<td>Immediate Use (IU)</td>
<td>Occasional with 40% probability of exceedance in 50 years (40/50)</td>
<td>Mean value of ( \theta_{y} ), may be exceeded by a factor of 2.0</td>
</tr>
<tr>
<td>Life Safety (LS)</td>
<td>Rare with 10% probability of exceedance in 50 years (10/50)</td>
<td>Safety factor ( \gamma^*<em>R ) of 1.35 against ( \theta</em>{pl}^{u,k} )</td>
</tr>
<tr>
<td>Collapse Prevention (CP)</td>
<td>Very rare with 2% probability of exceedance in 50 years (2/50)</td>
<td>( \theta_{pl}^{u,k} ) capacity may be reached (( \gamma^*_R = 1 ))</td>
</tr>
</tbody>
</table>

As discussed, seismic demands according to fib MC2010 should be calculated by rigorous nonlinear response-history analysis using lumped plasticity finite elements with bilinear moment-rotation hysteretic models and realistic rules for stiffness degradation during unloading and reloading.
Furthermore, the finite element model applied should use realistic estimates of the effective elastic stiffness of concrete members $EI_{\text{eff}}$ as given by Equation 3, where $M_y$ represents the end section yield moment and $L_s$ the shear span of the member on the side of the end section.

$$EI_{\text{eff}} = \frac{M_y L_s}{3\sigma_y}$$ (3)

2.5 Solution algorithms

Different algorithms can be used to solve the optimization problem of Equation 1. In this study, Genetic Algorithms (GAs) (Holland 1975) are applied. GAs belong to the class of stochastic, nature-inspired heuristic algorithms. They are based on Darwin’s theory of natural selection and evolution. GAs can be easily applied to optimization problems such as the one examined herein because they don’t require use of gradients of cost or constraints functions. Furthermore, they can identify global optima as opposed to local optimum solutions.

GAs iteratively modify populations (generations) of individuals to evolve toward an optimum solution. An individual $x$ (genome) represents a candidate solution to the optimization problem. The values of the design variables $x_i$ ($i=1$ to $n$) forming individuals are called genes. To create the next population, GAs select certain individuals in the current population (parents) and use them to create individuals in the next generation (children).

In this study, the mixed integer GA as implemented in MATLAB-R2017a (MathWorks 2017) 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 (Deep et al. 2009).

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.

2.6 Solution strategies

Application of efficient automated optimization algorithms is important in the solution of the optimization problem of Equation 1. However, all these algorithms have limitations and are not guaranteed to provide global optimum solutions in the case of complex problems with high number of design variables and enormous search spaces. Therefore, the adopted strategy in the formulation of the optimization problem is vital for its successful solution.

The first and simplest solution approach to the optimization problem of this study is to consider all the variables of the design vector $x$ as independent. In this case, the optimization algorithm selects all design variables of $x$. Then, the design solution is simply assessed by comparing the calculated seismic demands and limit values as shown in Table 1 for all Limit States. The drawback of this approach is that it requires a great number of design variables to be treated by the optimization algorithm that may become incapable of tracking the optimum solution.

The second solution strategy considers only $x_{cd}, x_{sl}$ sub-vectors as independent design variables. Knowing these vectors, the nonlinear finite element analysis model can be composed and the seismic demands are calculated. The design solution is then assessed for the SLS, where limit values depend solely on these sub-vectors. Next, the $x_{sw}$ is appropriately designed to meet seismic demands (both in terms of chord rotations and shear forces) for the ULS. This approach is typically used in literature dealing with optimum performance-based seismic design of reinforced concrete structures and has the advantage of reducing the independent design variables by removing the sub-vector $x_{sw}$ from the design vector $x$ in Equation 1. However, for reinforced concrete frames even of moderate complexity, this approach still requires a rather high number of design variables to be handled by the optimization algorithm.

To address this issue, a new solution methodology is proposed herein. The main feature of the proposed approach is that it employs a simple, deformation-based, iterative procedure for the design of steel reinforcement of reinforced concrete frames to meet their performance objectives given the cross-
sectional dimensions of their structural members. In this manner, only the cross-sectional dimensions of structural members need to be set as design variables reducing greatly the search space of the optimization problem and facilitating the optimization algorithms to reach the optimum solutions. Following this approach, the optimizer selects only the \( x_d \) design sub-vector. Then, the \( x_d \) is first design to resist static loads. The corresponding \( M_y \), \( \theta_y \) and \( E_{d_{eff}} \) values for all member ends are evaluated and the nonlinear finite element is composed. Seismic demands in terms of chord-rotations \( \theta_y \) are calculated for the SLS and checked against the corresponding capacities. In the cases where demands exceed the limit values, the required end section yielding moments \( M_{y,req} \) are estimated by Equation 4 and the longitudinal reinforcement is designed to achieve these moments. This iterative procedure continues until all SLS checks are verified and the \( x_d \) sub-vector is established. Finally, the \( x_w \) is selected to satisfy the performance objectives of the ULS.

\[
M_{y,req} = M_y \cdot \max \left\{ \left( \frac{\theta_y}{\theta_{y,lim}} \right)_{OP}, \left( \frac{\theta_y}{\theta_{y,lim}} \right)_{I,U} \right\} \tag{4}
\]

The proposed solution strategy reduces greatly the number of the independent design variables to be treated by the optimizer at the expense of a simple iterative procedure for the determination of the longitudinal steel reinforcement. It will be shown that it is more computationally efficient than the second solution strategy driving to more robust and less costly optimum solutions.

3. OPTIMUM SEISMIC DESIGN APPLICATIONS

3.1 Comparisons with prescriptive seismic design methodologies

In this section, a four-storey two-bay reinforced concrete frame (Figure 1) is optimally designed according to EC8 and MC2010 provisions (Mergos 2017a). 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.

The frame consists of 12 columns and 8 beams. Due to symmetry, it is assumed that the two exterior columns have 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. Using the first solution strategy, as described in §2.6, 23 independent design variables are employed for this problem.

For the optimum designs, it is assumed that section dimensions take values from the following discrete set: (0.25m; 0.30m; 0.40m; ... ; 1.5m). Furthermore, longitudinal bar diameters are defined in the following discrete values set: (12mm; 16mm; 20mm; 25mm). Transversal bars diameters take values from: (8mm; 10mm; 12mm). Transverse reinforcement spacing may take the following values: (0.1m; 0.15m; 0.20m; 0.25m; 0.30m). Finally, numbers of main bars and legs of shear reinforcement may take any integer value greater than one.

The frame is a part of a building of ordinary importance that rests on soil class B according to the classification of EC8. It is designed for 0.16g, 0.24g and 0.36g peak ground acceleration values for the 10/50 seismic hazard level to examine the influence of the level of seismicity (low, moderate and high respectively) on the optimum seismic design solutions. The 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 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. 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 may influence the cost of the optimal design solutions with respect to EC8 ground motion selection provisions. 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 and Figure 2b based on the MC2010 guidelines.

**Figure 1.** Examined four-storey two-bay frame

![Figure 1](image1.png)

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

![Figure 2](image2.png)

Figure 3 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. 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. 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.
Figure 4 presents MC2010 checks of rotation and shear force constraints (Equation 2) 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. 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.
It is evident 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 is seen that the rotation constraints of beam members
are in many cases close to zero, which means that they dominated this design solution. MC2010 design
for the THB motion set provides generally conservative results.

![Graph showing optimum costs](image)

Figure 3. Optimum costs obtained by different design methodologies and design PGAs a) in Euros; b)
percentile contributions
Figure 4. 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

3.2 Application of the proposed solution strategy

In this section, application of the proposed optimum seismic design methodology described previously to a reinforced concrete frame is presented. The goal is to compare the computational performance of the different solution strategies and investigate the applicability of the proposed approach. A six-storey two-bay reinforced concrete frame (Figure 5) is optimally designed. Frame spans are 5m and storey heights 3m. Concrete C25/30 and reinforcing steel B500C are used. Concrete cover is assumed to be 30mm. Storey weight for the quasi-permanent combination is 576kN and it is applied in the form of vertical point loads of 144kN at the exterior and 288kN at the locations of the interior columns. The frame is part of a building of ordinary importance that rests on soil class B. It is designed for 0.24g peak ground acceleration (PGA) for the 10/50 seismic hazard level. Based on the previous information, the Eurocode 8 – Part 1 (CEN 2004) elastic pseudo-acceleration response spectrum for regions of high seismicity is composed, as shown in Figure 6b, that serves as the target response spectrum in this study. The frame is designed to resist the Tabas 1978 earthquake ground motion recorded at station ST59 in the X direction scaled in amplitude so that its elastic response spectrum is not lower than 90% of the target response spectrum. The scaled acceleration time history and the corresponding response spectrum together with the Eurocode 8 elastic spectrum are shown in Figure 6.

For simplicity, square sections are assumed for the column members. Furthermore, column cross-sectional dimensions do not change along frame height. The steel reinforcement is assumed uniform along the length of column members, but may change every second storey. Beam cross-sectional heights can change every two storeys. The same beam width is assumed for all storeys that cannot be larger than the column cross-sectional dimensions for construction purposes. Different steel reinforcement is used for the exterior and interior supports of the beam series. The same top and bottom steel reinforcement is assumed for the beam members due to symmetry.

In addition to the previous, it is assumed herein that column cross-sectional dimensions and beam heights take values starting from 0.30m and increasing by 0.05m up to 1.00m. Beam width is assumed to take one of the following values: 0.30m, 0.35m and 0.40. Longitudinal bar diameters of 16mm and transverse bar diameters of 8mm are used for both beam and column members. Transverse reinforcement spacing takes values starting from 0.10m and increasing by 0.025m. The number of
longitudinal steel bars and legs of the transverse reinforcement can take any integer value between 2 and 10 for both beam and column members. Following these assumptions and due to symmetry, 6 different column sections and 6 beams sections are required. In total, 6 independent design variables are required by the proposed solution strategy (2 column cross-sectional dimensions, 3 beam heights and 1 beam width) and 18 design variables by the second (named standard) solution strategy (cross-sectional dimensions plus the numbers of longitudinal bars of the 6 beam and 6 column sections) as described in §2.6.

Figure 5. Reinforced concrete six-storey two-bay frame

![Figure 5](image)

Figure 6. Scaled Tabas 1978 earthquake ground motion a) accelerogram; b) response spectrum

![Figure 6](image)

Figure 7a compares the minimum costs obtained from all GA analyses for both solution strategies. The proposed strategy yields an optimum solution with 21% (5391€ instead of 6820€) less cost than the standard approach. In addition, Figure 7b illustrates the minimum costs obtained from 10 different GA runs for both solution methodologies in the form of box plots. The box plots show the minimum, maximum and median (red line) minimum costs obtained from the 10 GA runs. Inside the boxes, the 25th to 75th percentile solutions are contained. It is shown that the mean minimum cost prediction of the proposed approach is 28% less expensive than the standard solution. It is noted that the best prediction of the standard approach is significantly worse than the worst prediction of the proposed methodology.
Moreover, the variability of the proposed method is smaller than the standard approach (i.e. Coefficient of Variation 3.5% instead of 6.0%).

Figure 7. Comparison of the computational performance of the standard and proposed solution strategies for the 6-storey 2-bay reinforced concrete frame

Furthermore, Figure 8 presents rotation and shear force constraints checks of MC2010 for the two optimum solutions of Fig. 7a. Only the checks that contain one constraint value greater than -0.25, either for the standard or the proposed solution strategy are shown for illustration reasons. As expected, all constraints checks are satisfied and the optimum solutions are acceptable. It is also noteworthy that in
many cases the constraints are very close to the limit zero value, which shows the efficiency of the derived optimum solutions.

4. CONCLUSIONS

Performance- and deformation-based seismic design is the most effective seismic design approach to control the levels of seismic damage. In performance-based seismic design, it is vital that the seismic demands are predicted reliably. Clearly, the most accurate method for determining seismic demands is the inelastic response history analysis with step-by-step integration of the equation of motion in the time domain.

Until this point, the number of applications of automated structural optimization techniques to the performance- and deformation-based seismic design of reinforced concrete frames with the aid of nonlinear response-history analysis is rather limited. This can attributed to the high levels of required computational effort and the great number of design variables involved in the seismic design of reinforced concrete frames.

First in this study, a comparison is made between optimum seismic designs obtained by the performance-based seismic design methodology of MC2010 using nonlinear response-history analysis and optimum designs following the prescriptive seismic design methodology of EC8 for different ductility classes. It is found that the performance-based seismic design methodology leads to more cost-effective structural solutions in regions of low to moderate seismicity. In regions of high seismicity, the minimum costs of the performance-based and the prescriptive seismic design methodologies are similar. In all cases, the MC2010 approach yield more reliable designs by controlling better seismic damage for different levels of seismic hazard.

In addition, a new computationally efficient solution strategy is proposed herein to address automated optimum performance-based seismic design of reinforced concrete frames. The main feature of the proposed approach is that it employs a simple, deformation-based, iterative procedure for the design of steel reinforcement of reinforced concrete frames to meet their performance objectives given the cross-sectional dimensions of their structural members. In this manner, only the cross-sectional dimensions of structural members need to be set as design variables reducing greatly the search space of the optimization problem and facilitating the optimization algorithms to reach the optimum solutions. The proposed seismic design methodology is applied to the performance-based seismic design of a reinforced concrete frame. It is shown that it produces more robust and cost-effective optimum design solutions than the standard approaches.

5. REFERENCES


