Minimum cost performance-based seismic design of reinforced concrete frames with pushover and nonlinear response-history analysis

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Abstract. Previous studies compare results of pushover and nonlinear response-history analysis of pre-designed reinforced concrete frames. The present study employs nonlinear response-history analysis and pushover analysis with the N2 method in a computational framework for the minimum cost performance-based seismic design of reinforced concrete frames according to the *fib* Model Code 2010 methodology and compares their obtained design solutions in terms of cost and structural performance. It is found that the minimum costs of pushover-based designs are similar to the costs from response-history analysis for regular frames but the pushover-based designs can be more expensive for irregular frames. Furthermore, the pushover-based designs are not guaranteed to satisfy performance objectives when subjected to response-history analysis even when more than one lateral load distributions are applied.

Keywords: Optimization; deformation-based; performance-based; seismic design; reinforced concrete; pushover; response-history; genetic algorithms; *fib* Model Code 2010

Running Head: Seismic design with pushover and response-history analysis

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1 Introduction

Accurate determination of seismic demands is a key in successful performance- and deformation-based seismic design. Without any doubt, nonlinear response-history analysis is the most accurate method for calculating seismic demands. Nevertheless, pushover analysis can also be applied to skip the complexity and computational requirements of nonlinear response history analysis.

Despite its simplicity, pushover analysis provides important information on various aspects of structural response that cannot be obtained by linear elastic and dynamic analysis (Krawinkler, 1996; Antoniou and Pinho, 2004a). However, this structural analysis procedure exhibits several limitations, mainly related to its inability to account for local damage accumulation, higher mode effects and variation of modal characteristics due to inelastic response (Krawinkler, 1996; Antoniou and Pinho, 2004a; Kalkan and Kunnath, 2007). To overcome these drawbacks, several enhanced pushover procedures have been proposed (e.g. Chopra and Goel, 2002; Antoniou and Pinho, 2004b; Kalkan and Kunnath, 2006; Kreslin and Fajfar, 2011, Bergami *et al*. 2017) exhibiting different trade-offs between reliability and computational efficiency.

Research efforts associated with the automated optimum performance- and deformation-based seismic design of reinforced concrete (RC) structures are rather limited. This deficit can be attributed to the large number of design variables required for RC structures (Sarma and Adeli, 1998) and the computationally intensive nonlinear structural analysis procedures needed to calculate seismic demands. An indicative list of these studies that is not aiming at being exhaustive is given below with a focus on the structural analysis procedures employed to determine seismic demands.

Ganzerli *et al*. (2000) employed pushover analysis to conduct optimum performance- and deformation-based seismic design of a portal frame. Chan and Zou (2004) developed a procedure for the optimization of the seismic design of RC frames that consists of two phases. In the first phase, member section dimensions are chosen to fulfil the Serviceability Limit State for frequent earthquakes by employing linear elastic analysis. In the second phase, steel reinforcement is calculated to satisfy the Ultimate Limit State for rare earthquakes. In this phase, pushover analysis is used to calculate seismic demands. Lagaros and Papadrakakis (2007) used a multi-objective optimization framework to compare seismic designs based on Eurocode 8 (EC8) (CEN, 2004) and a performance-based approach. For the latter approach, pushover analysis was used to determine storey drifts. Fragiadakis and Papadrakakis (2008) formulated a framework for the optimum performance-based seismic design of RC frames employing nonlinear response-history analysis. Lagaros and Fragiadakis (2011) compared optimum seismic designs of RC building frames using three different pushover methods. These were the ATC-40 (ATC, 1996) capacity spectrum method, the displacement coefficient method of ASCE-41 (ASCE/SEI, 2006), and the N2 method (Fajfar, 2000) of EC8. Gencturk (2013) compared performance-based seismic designs of RC and ECC (Engineered Cementitious Composites) frames using a multi-objective optimization framework. Pushover analysis was employed to calculate structural capacities and response-history analyses to determine seismic demands in terms of storey drifts. Mergos (2017) compared optimum designs of RC frames following EC8 and the *fib* Model Code 2010 (MC2010) performance-based seismic design methodology (*fib*, 2012; Fardis, 2013). In this study, seismic demands were determined by nonlinear response-history analysis. Furthermore, Mergos (2018) developed a computationally efficient procedure for the optimum seismic design of RC structures where only cross-sectional dimensions are used as design variables. The developed solution strategy is applied to the optimum seismic design of reinforced concrete frames using pushover and nonlinear response-history analysis and it is found that it outperforms previous solution approaches. Gharehbaghi (2018) developed optimum designs of concrete frames for minimum cost with the additional constraint of uniform damage distribution over the height of the frames using nonlinear response history analysis with an artificial ground motion record. Furthermore, Gholizadeh and Aligholizadeh (2018) examined reliability-based optimum performance-based seismic design of RC frames with meta-models and metaheuristic algorithms by using pushover analysis to calculate seismic demands.

It is evident by the previous that either pushover or nonlinear response-history analysis is used to evaluate seismic demands in the context of optimum performance-based seismic design. In this study, reinforced concrete frames will be optimally designed by employing both structural analysis procedures in order to compare the obtained seismic design solutions. For pushover analysis, the effect of using different lateral load patterns will also be examined. The aim here is to examine whether the pushover-based optimum designs can control adequately the level of structural damage and how these designs are related in terms of cost to the optimum designs based on nonlinear response-history analysis. These questions are critical since the use of pushover analysis in the context of automated optimum performance-based seismic design, where numerous trial design solutions are examined, can reduce substantially the computational cost with respect to response-history analysis.

2. Optimum seismic design of RC frames problem formulation and solution

2.1 Optimization problem formulation

Optimum performance-based seismic design of RC frames can be formulated as a single-objective optimization problem with discrete design variables. This is the case because the design variables can take only either integer (e.g. numbers of steel reinforcement bars) or pre-determined discrete values (e.g. cross-sectional dimensions, bar diameters) prescribed by the construction industry. This optimization problem is generally written as:

Minimize: $$C(\boldsymbol{x})$$
Subject to: $$g_j(\boldsymbol{x}) \leq 0,\ j = 1\ to\ m \quad (1)$$
Where:
$$\boldsymbol{x} = (x_1, x_2, \ldots, x_n)$$

$$x_i \in D_i = \left(d_{i1}, d_{i2}, \dots, d_{ik_i}\right), i = 1 \; to \; n$$

In Eq. (1), $C(\boldsymbol{x})$ is the objective function to be minimized. The vector $\boldsymbol{x}$ represents the candidate design vector, which contains n number of independent design variables x_i (i =1 to n). The design variables x_i assume values from pre-specified sets of discrete values $\boldsymbol{D}_i = (d_{i1}, d_{i2}, \dots, d_{iki})$, where d_{ip} (p =1 to k_i) is the p-th possible discrete value of the design variable x_i and k_i is the number of all possible discrete values of x_i. In addition, the solution is subject to m number of constraints $g_j(\boldsymbol{x}) \leq 0$ (j =1 to m). In the following, the different features of the optimization problem are described in more detail.

Generally, the input data of an optimization problem are distinguished in design parameters that keep fixed values and design variables that change during the optimization solution. Herein, design parameters are assumed the geometry, material properties, concrete cover and loading of RC frames. Design variables determine section and steel reinforcement properties shown in Fig. 1 for beam and column sections. Seven design variables are used for the column sections (i.e. h_c, b_c, d_{bc}, n_c, d_{bwc}, n_{wc}, s_c), assuming symmetric reinforcement configuration, and nine design variables for beam sections (i.e. h_b, b_b, d_{bt}, n_{tb}, d_{bb}, n_{tb}, d_{bwb}, n_{wb}, s_b). A more detailed description of these design variables can be found in Mergos (2017). It is noted herein that, for simplicity, the present study is focusing only on concrete beam and column members and design of frame joints is not examined.

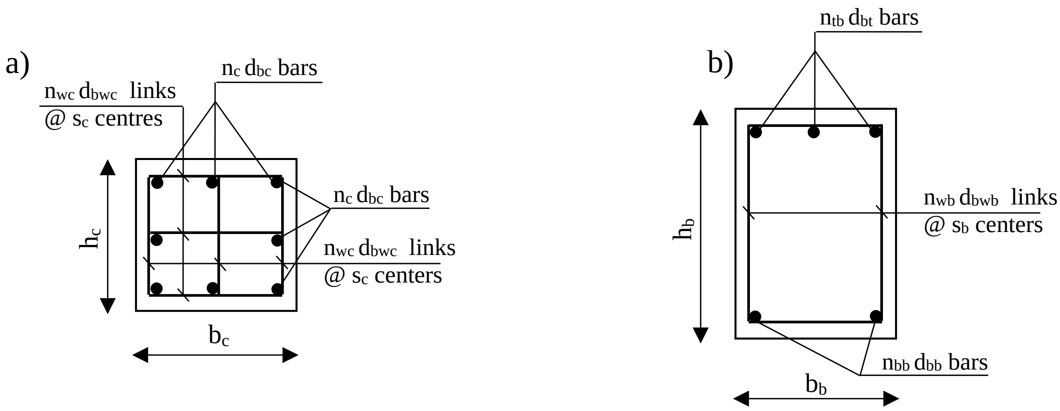


Fig. 1: Design variables: a) column sections; b) beam sections

The objective function $C(\boldsymbol{x})$ investigated herein is the material cost of RC frames. This cost can be taken as the sum of costs of concrete $C_c(\boldsymbol{x})$, steel $C_s(\boldsymbol{x})$ and formworks $C_f(\boldsymbol{x})$. The following unit prices are used in this study for calculating the respective costs: cost of concrete per unit volume C_{co}=100Euros/m^3, cost of steel per unit mass C_{so}=1Euro/kg and cost of formwork per unit area C_{fo}=15Euros/m^2. In a similar fashion but with different unit prices, other significant objectives functions can be considered in the optimization problem such as the embodied environmental impact (e.g. CO_2 emissions) of RC frames (Mergos, 2018).

Design constraints $g_j(\boldsymbol{x})$ in performance-based seismic design of RC frames can be divided into two main categories: Engineering Demand Parameter (EDP) constraints and Structural Design Parameter (SDP) constraints. The first category reflects the requirement that $EDPs$ (e.g. displacement, rotations, shear forces) must remain below a limit value EDP_{cap}. These constraints can be written in the normalized form of Eq. (2). The second category represents the requirement that structural detailing parameters (e.g. sectional dimensions, bar diameters, steel reinforcement ratios) should be smaller or greater than limit values specified by design regulations.

$$g_j(\boldsymbol{x}) = \frac{EDP}{EDP_{cap}} - 1 \leq 0,\ j \in (1, 2, \dots, m) \tag{2}$$

In this study, the design constraints are set in accordance with the performance-based seismic design methodology of MC2010 for seismic loads and following the specifications of Eurocode 2 (EC2) (CEN, 2000) for static loads. MC2010 examines four discrete Limit States (*fib*, 2012; Fardis, 2013). The Operational (OP) and Immediate Use (IU) are Serviceability Limit States (SLS), whilst the Life Safety (LS) and Collapse Prevention (CP) represent Ultimate Limit States (ULS). All Limit States are expressed in terms of chord rotations and checked for different levels of Seismic Hazard. More particularly, the SLS are expressed in terms of yield chord rotations θ_y and the ULS in terms of characteristic plastic chord rotation capacities $\theta^{pl}_{u,k}$ of structural members. Additionally, shear failures are checked in terms of shear forces for the two ULS. A detailed description of the applied design constraints and how they are applied in the optimization framework of this study can be found in Mergos (2017).

2.2 Optimization problem solution

In the previous section, the adopted design variables are described. It is easily understood that even for simple RC frames a significant number of design variables is required increasing significantly the size of the search space and undermining the ability of tracking the global optimum solutions. To limit this issue, the design variables can be classified as primary and secondary. The former are selected directly by the optimizer whilst the latter are then specified based on the primary ones.

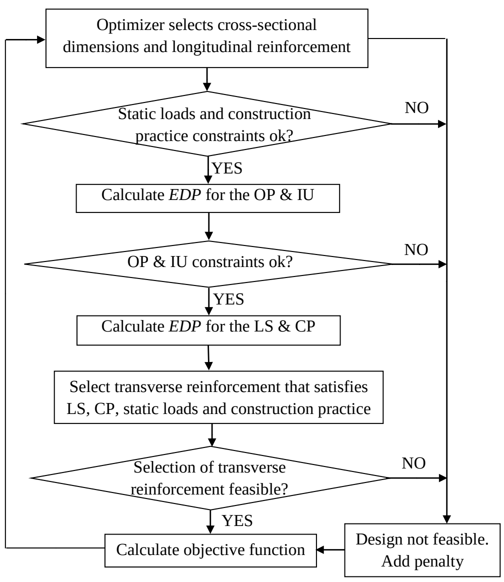


Fig. 2: Optimization problem solution strategy

Herein, primary variables are the ones associated with the sectional dimensions and the longitudinal steel reinforcement of the RC members. Knowing them is adequate to conduct nonlinear structural analyses and obtain the corresponding *EDP*s values for different levels of Seismic Hazard as well as calculating θ_y rotations that specify SLS constraints. Based on the obtained *EDPs* values, the secondary design variables related to the transverse steel reinforcement of structural members are explicitly determined to fulfil the performance requirements expressed in terms of plastic chord rotations and shear forces of the ULS.

In the cases where the SLS constraints are not fulfilled or selecting transverse steel reinforcement to satisfy ULS constraints is not feasible then the candidate design is branded unfeasible and a penalty value is added to the objective function. The recommended solution strategy is shown in Fig. 2.

Different optimization algorithms can be used to address the problem examined herein as long as they can treat discrete design variables. In this study, the mixed integer GA (Holland, 1975) included in MATLAB-R2017a (MathWorks, 2017) is used. This GA treats both discrete and continuous design variables by employing special mutation and crossover functions (Deep *et al.*, 2009). Moreover, it handles nonlinear constraints by applying the penalty function approach (Deb, 2000).

3. Optimum designs of RC frames with pushover and response-history analysis

3.1 Introduction

In this section, comparisons of optimum performance-based seismic design solutions of RC frames obtained by using either pushover or response-history analysis are presented. A regular frame as well as a concrete frame with setbacks are examined. The frames are parts of buildings of ordinary importance that rest on soil class B following the classification of EC8. The frames are designed for 0.36g PGA for the 10/50 Seismic Hazard level. PGAs for the other levels of seismic hazard are calculated by multiplying the reference 10/50 values by the importance factor γ_I as specified in EC8.

All frames are designed according to MC2010 performance-based seismic design methodology. However, as discussed and in order to serve the purpose of this study, *EDPs* are calculated either by nonlinear response-history analysis or pushover analysis. In the following, the design solutions based on nonlinear response-history analyses are designated as TH. The nonlinear response-history analyses are carried out with the aid of computer software IDARC2D (Reinhorn *et al*. 2009) using the Newmark constant acceleration integration algorithm. The one-component lumped plasticity finite element (Giberson 1967) is used in this study for calculating seismic demands as suggested in MC2010. This is a series model of an elastic element and two nonlinear rotational springs at its ends, where all inelastic deformations are lumped. The effective stiffness of the elastic element and the envelopes of the nonlinear springs are evaluated based on the recommendations of MC2010. Furthermore, hysteretic rules representative of well-detailed and flexure-controlled reinforced concrete members with mild stiffness degradation during unloading and reloading are applied following the recommendations by Sivaselvan and Reinhorn (1999) and Mergos and Kappos (2012).

The nonlinear response-history analyses are conducted for a set of seven ground motion records selected from the European Strong-Motion Database (ESD) (Ambraseys *et al*. 2002) and scaled by the computer program REXEL (Iervolino *et al*., 2009) so that their mean elastic spectrum follows closely the elastic (target) spectrum of EC8 for high and moderate seismicity regions, as illustrated in Fig. 3 for the case of the 10/50 Seismic Hazard level. The main characteristics of the selected ground motions are shown in Table 1.

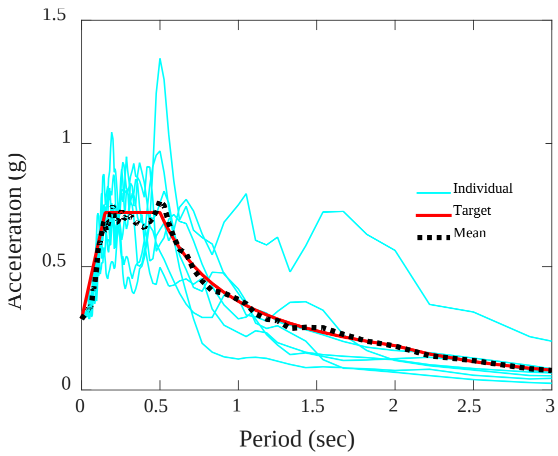


Fig. 3: Scaled elastic spectra with 5% damping of selected set of ground motions for the 10/50 Seismic Hazard level

In addition, the RC frames are designed with the aid of pushover analysis following the N2 method (Fajfar, 2000) as prescribed in EC8 (CEN, 2004). This is a relatively straight-forward method that links the pushover analysis of a MDOF model with the response spectrum analysis of an equivalent SDOF system. It is worth noting that the N2 method has been incorporated in EC8, where it can be used either to assess the seismic response of existing structures or to verify the over-strength ratio influencing the behaviour factor in the design of new structures. The same finite element model and computer software are used to run the pushover analyses as the ones used in this study for conducting nonlinear response history analyses.

Three different applications of pushover analysis are examined herein. In the first case, designated as PUS1, the design solutions are checked by pushover analysis with a ‘uniform’ load pattern, where lateral forces are proportional to storey masses. In the second case, designated as PUS2, the frames are verified by pushover analysis with a ‘modal’ load pattern, where lateral forces are consistent with the lateral force distribution determined in elastic analysis. In the third case, designated as PUS3, the frames are checked by conducting pushover analysis with both a ‘uniform’ and a ‘modal’ load pattern. The most onerous *EDP* demands of the two load patterns are used to examine the design constraints. The latter approach is recommended in EC8 to account for the uncertainties in the distribution of lateral loads for structures due to higher mode effects and inelastic response.

Table 1: Selected ground motion records

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

For the optimum designs, it is assumed that section dimensions h_c, b_c, h_b, b_b take values multiples of 50mm starting from 300mm. Numbers of main bars n_c, n_{tb}, n_{bb} and legs of shear reinforcement n_{wc} and n_{wb} take any integer value greater than one. Transverse reinforcement spacing s_c and/or s_b take values between 100mm and 300mm with a step of 25mm. Longitudinal bar diameters d_{bc}, d_{bb}, and d_{bt} and transversal bar diameters d_{bwc}, and d_{bwb} are given fixed, pre-specified values for each RC frame in order to reduce the number of design variables and therefore facilitate the task and increase the accuracy of the optimization algorithm.

In general, the default options and parameter values of the MATLAB GA algorithm (MathWorks, 2017) are adopted in this study. The default GA population size according to (MathWork, 2017) is equal to min(max(10·*nvars*, 40), 100), where *nvars* is the number of design variables. In all cases, GA iterations are terminated when the mean relative variation of the best fitness value is negligible for 100 generations. For each optimum design problem, 10 independent GA runs are performed to account for the stochastic nature of the GA algorithms and the minimum cost solution is typically reported. It is found that, in every case, the 10 independent GA runs yield very similar results with the coefficient of variation of the minimum costs obtained not exceeding 1%. In the following, the examined RC frames and the main obtained results are presented.

3.2 Three-storey two-bay frame

In this section, a three-storey two-bay frame representative of regular low-rise buildings is examined (Fig. 4). The span of the frame is 4m and storey height is 3m. Concrete C25/30 and reinforcing steel B500C are used. Concrete cover is assumed to be 30mm. Vertical point loads of 144kN at the exterior and 288kN at the interior joints are applied to represent seismic weight. No additional distributed loads are applied to the beams. Due to symmetry, end columns are assumed to have the same section and the beams the same top and bottom longitudinal reinforcement. For simplicity, square column sections are used and all beams are assumed to have the same section with fixed width of 0.30m. It is also assumed that the longitudinal and transverse steel reinforcement do not vary along the length of the members.

The diameter of the longitudinal bars is set to be 16mm and of the transverse reinforcement 8mm. Based on these design assumptions, one beam and two column sections are applied as shown in Fig. 4 with 6 primary design variables.

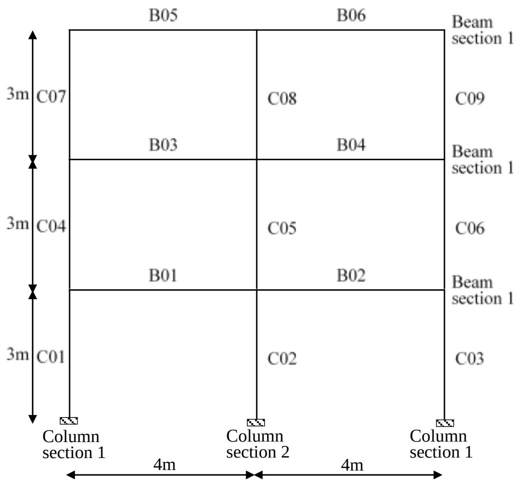


Fig. 4: Three-storey two-bay frame

Fig. 5 presents the optimization histories and the costs of the optimum design solutions of the examined frame designed by employing response-history (TH) and pushover analysis with “uniform” (PUS1), “modal” (PUS2) and both “uniform” and “modal” load patterns (PUS3). It can be deducted that the optimum costs obtained by using pushover analysis with only one load distribution are 6-8% smaller than the solution based on response-history analysis. On the other hand, the pushover-based optimum design with both load distributions has almost the same cost as the TH design. This also means that the PUS3 design is 6-8% costlier than PUS1 and PUS2 respectively. The significant increase in the cost is justified by the fact that the PUS3 design needs to satisfy *EDP* constraints for two different lateral load distributions and therefore it is more conservative.

The details of the design solutions are shown in Table 2. In this table, ρ_l is the volumetric ratio of the longitudinal reinforcement and ρ_w represents the volumetric ratio of the transverse that is parallel to the applied shear force. It is interesting to observe that all pushover-based solutions use similar cross-sectional dimensions. However, these dimensions are quite different than the TH solution. This is despite the fact that the TH and PUS3 solutions exhibit very similar costs.

Figure 6 presents MC2010 constraints checks of all optimum designs when subjected to response-history analysis with the set of ground motions in Table 1. In these figures, column sections are specified by the column number (e.g. C1) followed by a letter designating the location of the section in the member (i.e. T=top, B=bottom). Similarly, beam sections are specified by the corresponding beam member number (e.g. B1) and a letter designating the location of the section in the member (i.e. R=right, L=left). All specified sections designations are followed by the acronym of the Limit State for which the performance check is conducted. For clarity, only the checks with values greater than -0.25 for at least one design solution are presented. As expected, the TH design experiences no violations for all performance constraints. On the other hand, all pushover-based designs exhibit significant violations of the rotation constraints at the base of column C02. Minor violations are also observed for the beam rotations constraints of the PUS1 and PUS2 design solutions but not the PUS3 solution. No violations of the shear forces constraints are noticed for all pushover-based designs.

Furthermore, Fig. 7a presents the displacement time histories at the top of the TH and PUS3 optimum frames when subjected to the Aigion ground motion of Table 1 scaled to match the target spectrum of Fig. 3 for the 10/50 Seismic Hazard level. This ground motion was found to be the most damaging for the two frames under investigation. Top displacements are good indicators of the global frame responses. It is shown that the two frames have very similar maximum top displacement demands. On the other hand, Fig. 7b shows moment versus plastic rotation local responses at the base of column C02 for the same ground motion. It is evident that the PUS3 solution develops significantly larger plastic rotation responses than the TH solution. This explains why the corresponding rotational constraint is not satisfied for this frame and demonstrates that RC frames similar global responses do not ensure similar local responses.

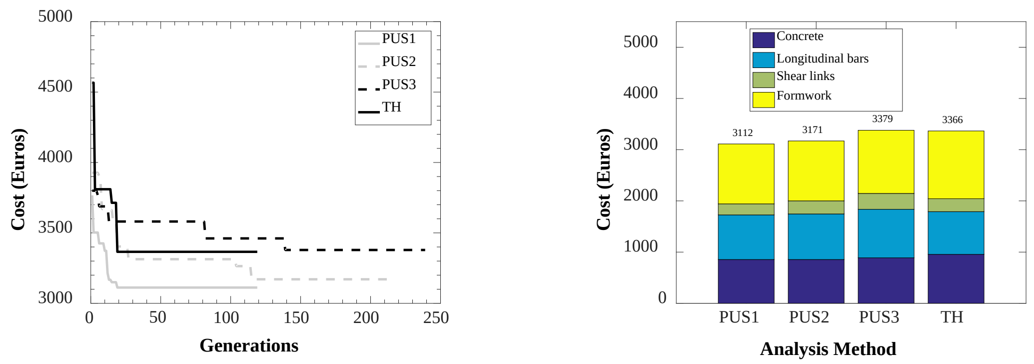
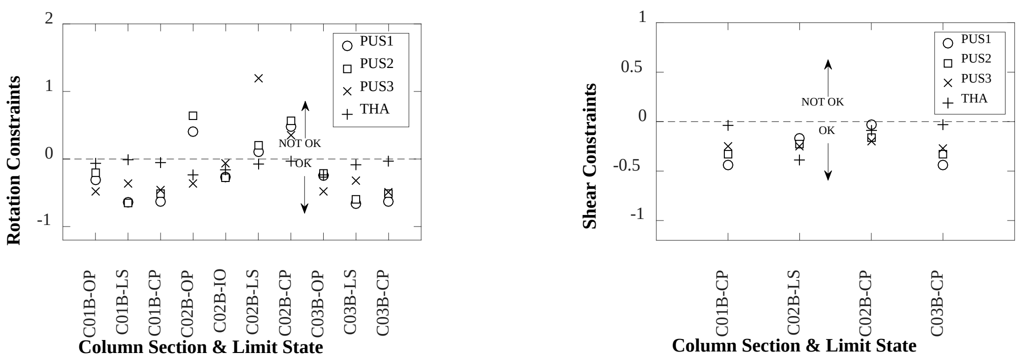


Fig. 5: a) Optimization histories; b) Costs of optimum design solutions of the three-storey two-bay frame

Table 2: Properties of optimum design solutions of the three-storey two-bay frame

Members	**Columns**								**Beams**			
Sections	**Section 1**				**Section 2**				**Section 1**			
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
	m	m	%	%	m	m	%	%	m	m	%	%
PUS1	0.3	0.3	1.79	0.19	0.7	0.7	0.98	0.22	0.35	0.3	1.53	0.15
PUS2	0.3	0.3	1.79	0.19	0.7	0.7	0.82	0.25	0.35	0.3	1.91	0.15
PUS3	0.35	0.35	1.31	0.16	0.65	0.65	1.14	0.31	0.4	0.3	1.67	0.22
TH	0.5	0.5	0.64	0.17	0.45	0.45	1.59	0.33	0.45	0.3	1.49	0.19



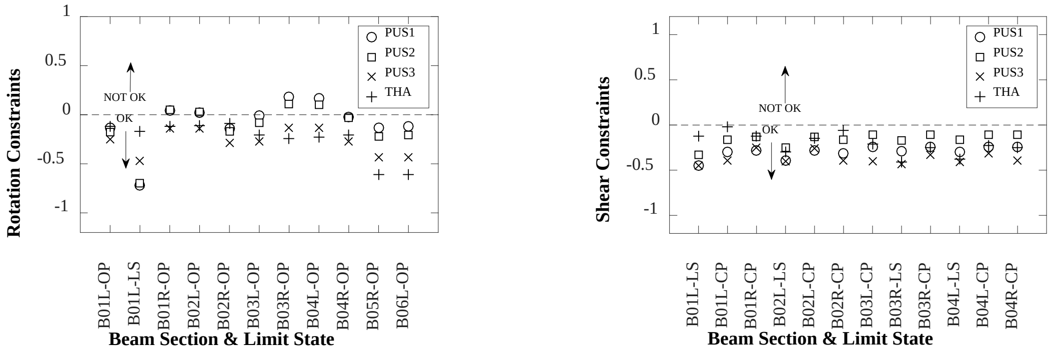


Fig. 6: Constraints of the optimum solutions of the three-storey two-bay frame obtained by response-history and pushover analysis

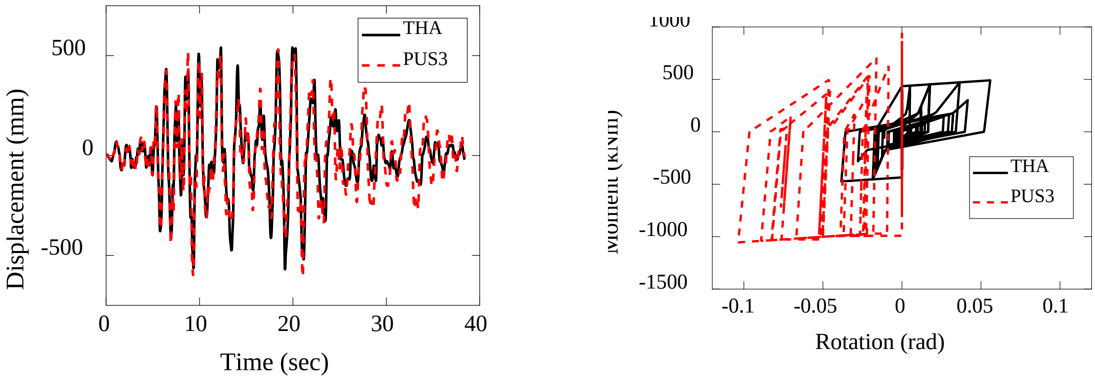


Fig. 7: a) Top displacement histories; b) Moment vs plastic rotation hysteretic responses at the base of column C02 of the PUS3 and TH optimum frames subjected to the scaled Aigion ground motion for the 10/50 Seismic Hazard level

3.3 Four-storey frame with setbacks

In this section, a four-storey frame (Fig. 8) with setbacks representative of irregular frame buildings in elevation is examined. The span of the frame is 4m and storey height 3m. Concrete C25/30 and reinforcing steel B500C are used. Concrete cover is assumed to be 30mm. Vertical point loads of 144kN at joints are applied at the exterior and of 288kN at the interior joints are applied to account for seismic weight. No additional distributed loads are applied to the beams. Two different column sections are used. One for the exterior and one for the interior columns. Furthermore, two beam sections are used. The first beam section is used in the first two storeys and the second beam section in the last two storeys. For simplicity, square column sections and a fixed beam width of 0.30m are assumed. The diameter of the longitudinal bars is assumed to be 16mm and of the transverse reinforcement 8mm. In total, 8 primary independent design variables are used for the optimum design of this frame.

Fig. 9 presents the optimization histories and the costs of the optimum design solutions of the examined frame designed by using response-history and pushover analysis. It is found that in this case the costs of the optimum designs obtained by using pushover analysis range quite significantly. More specifically, they range from 79% (PUS1) to 105% (PUS3) of the TH solution. The use of two load distributions (PUS3) increases the optimum cost by 19% with respect to the most expensive load distribution (PUS2).

In Table 3 the detailing characteristics of the optimal solutions are presented. It can be observed that the pushover-based designs differ significantly between them. Interestingly, their cross-sectional dimensions envelope the dimensions of the cross-sections of the TH solution. The PUS3 is the pushover-based design closest to the TH solution.

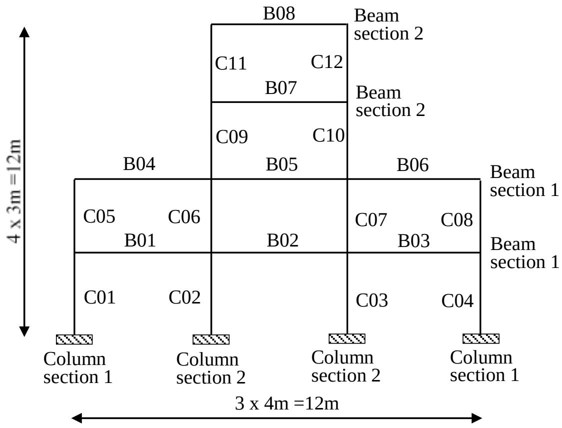


Fig. 8: Four-storey frame with setbacks

Table 3: Properties of optimum design solutions of the four-storey frame with setbacks

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
	m	m	%	%	m	m	%	%	m	m	%	%	m	M	%	%
PUS1	0.6	0.6	0.11	0.10	0.35	0.35	0.98	0.16	0.35	0.3	0.77	0.15	0.3	0.67	0.13	0.3
PUS2	0.5	0.5	0.16	0.11	0.35	0.35	0.98	0.34	0.55	0.3	0.97	0.15	0.45	0.3	1.49	0.27
PUS3	0.6	0.6	0.33	0.19	0.45	0.45	0.60	0.22	0.35	0.3	1.91	0.15	0.85	0.3	0.79	0.11
TH	0.55	0.55	0.53	0.10	0.45	0.45	0.60	0.18	0.4	0.3	1.34	0.13	0.6	0.3	1.34	0.33

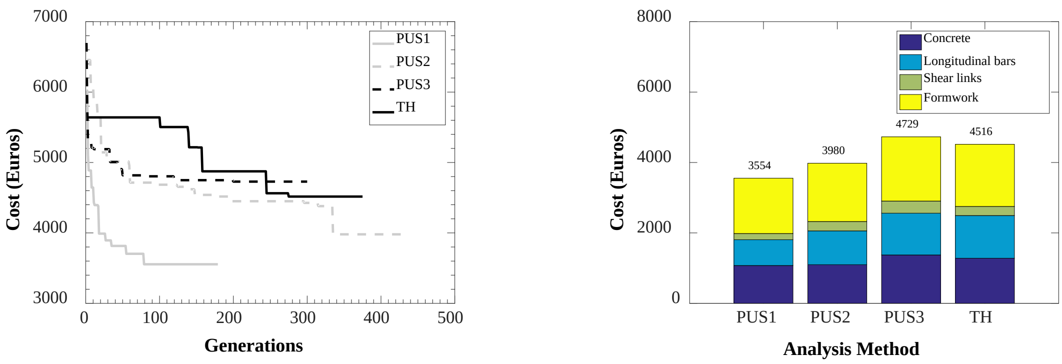


Fig. 9: a) Optimization histories; b) Costs of optimum design solutions of the four-storey frame with setbacks

Figure 10 presents MC2010 constraints checks of all optimum designs when assessed with the set of ground motions in Table 1. For clarity, only the checks with constraints violations for at least one design solution are presented. Numerous violations of the beam and column rotation constraints for the pushover-based designs are observed. These occur mainly at the bottom of the frame and at the location of the setbacks. Furthermore, the PUS3 design solution presents violations of the shear forces constraints at the beams of the setbacks. This clearly shows the disadvantages of pushover analysis when applied to irregular structures.

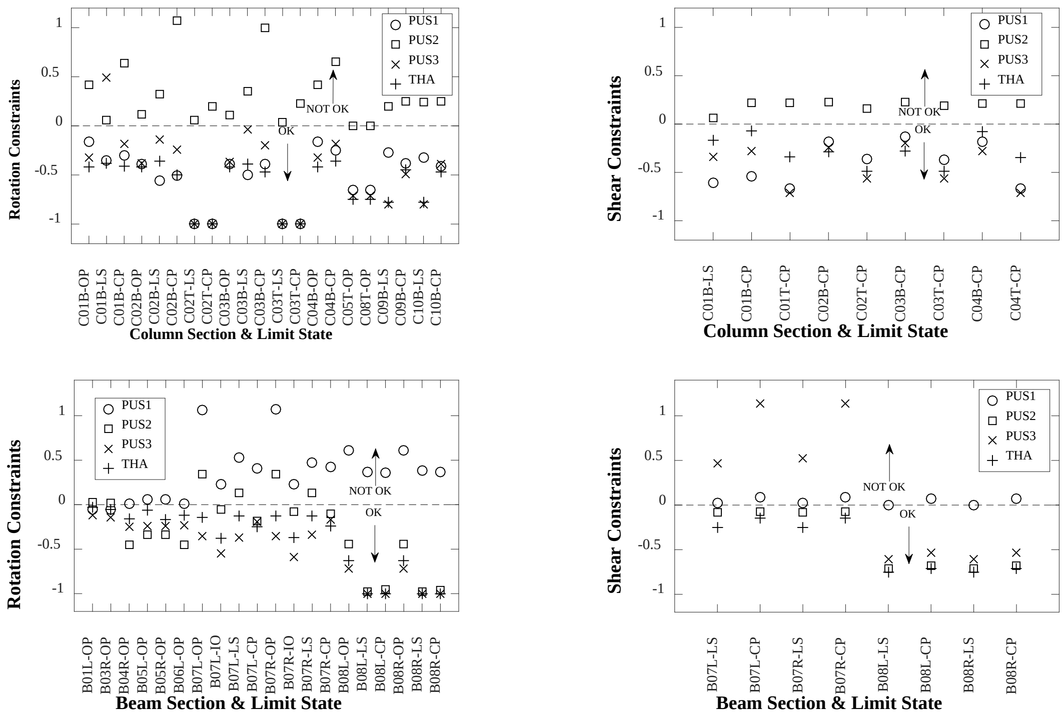


Fig. 10: Constraints of the optimum solutions of the four-storey frame with setbacks obtained by response-history and pushover analysis

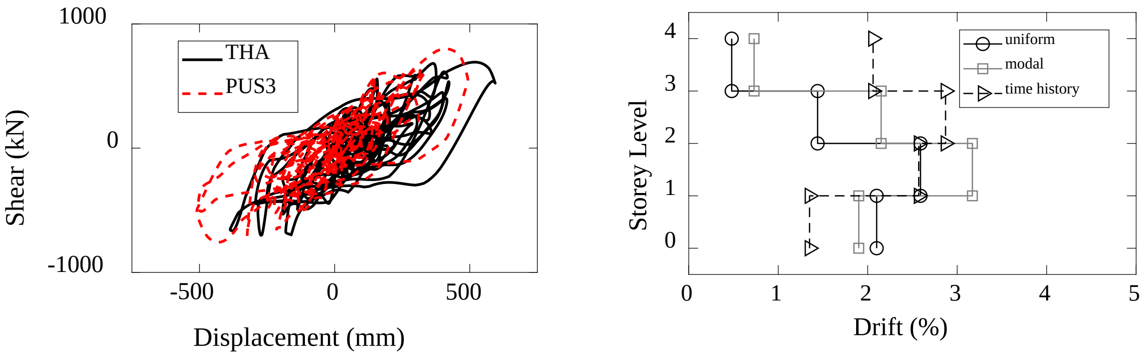


Fig. 11: a) Base shear vs top displacement response of THA and PUS3 optimum frames; b) Maximum drifts of frame PUS3 obtained by pushover and time-history analysis

Figure 11a shows the top displacement vs base shear responses of the TH and PUS3 optimum frames when subjected to the Aigion ground motion of Table 1 scaled to match the 10/50 target spectrum. It is seen that the two frames exhibit rather similar global responses. Furthermore, Fig. 11b presents the average maximum inter-storey drifts of the PUS3 optimum frame as predicted by time history analysis using the seven records of Table 1 and by the two pushover lateral load distributions for the 10/50 Seismic Hazard level. It is evident that the results obtained are significantly different both at the bottom of the frame and the locations of the setbacks. This further shows the inability of pushover methods to predict local responses of irregular frames and explains why the pushover-based designs are not able to satisfy constraints when subjected to time history analyses.

4 Summary and Conclusions

In the existing literature, pushover and nonlinear response-history analysis procedures are compared only as tools for the assessment of seismic demands of existing structural designs. This study compares these two structural analysis procedures as integral parts of automated, optimum performance-based seismic design of new structures. The goal here is to investigate if optimum seismic design using pushover analysis can control adequately the level of structural damage and how its cost is related to the cost of optimum design based on nonlinear response-history analysis. These questions are critical because the use of pushover analysis in the context of automated optimum performance-based seismic design can reduce substantially the computational cost.

To serve this goal, a general optimisation framework for the design of RC frames is employed that is based on genetic algorithms and complies fully with the performance-based seismic design methodology of *fib* Model Code 2010. The framework is applied to the design of RC frames with different structural configurations using nonlinear response-history and pushover analysis as prescribed by the N2 method in EC8. The pushover analysis is conducted by assuming “uniform”, “modal” and both “uniform” and “modal” invariant lateral load distributions.

It is found that the costs of the optimum designs based on pushover analysis depend on the applied lateral load distribution and that the use of more than one lateral load distributions increases the optimum costs. For low-rise, irregular in elevation, frames the cost of optimum designs increases by 19% when two load distributions are applied instead of one. However, for regular low-rise frames the differences are considerably smaller.

Furthermore, it is observed that the costs of the optimum pushover-based designs with two load distributions are generally close to the ones obtained by nonlinear response-history analysis with the differences ranging up to 5%. The differences are more important in the case of irregular frames.

In terms of damage control, it is found that the pushover-based designs, using the N2 method as prescribed in EC8, are not always guaranteed to satisfy fully local performance requirements when subjected to response-history analysis even when more than one load distributions are used. The extent of the violations of the performance constraints was found to be rather limited in the case of regular frames. However, it became more important for irregular frames. More advanced pushover methodologies or more conservative solutions could be used to address this limitation.

Nevertheless, the pushover-based designs represent reasonable approximations of the optimum design solutions from response-history analysis. Hence, they could be used as good starting points in the search of optimum solutions based on response-history analysis.

In light of the previous findings, the use of pushover analysis with more than one lateral load distributions in the optimum, performance-based seismic design of low-rise regular frames is worth further consideration as it reduces grossly the computational costs and it does not increase considerably the material costs. However, this approach should be accompanied by additional measures aiming at increasing the reliability of the pushover method at the local responses level.

The present study focused on plane, low-rise RC frames that are regular or irregular in elevation. Further studies are required to examine the applicability of the pushover method to the optimum seismic design of medium- and high-rise RC frames as well as 3-dimensional frames that are symmetric or asymmetric in plan.

It is also important to note that the current study considered materials cost as the single objective to be minimized by the optimization solution. Additional considerations are required to account for labour costs during construction. Moreover, further research is required to examine the effects of the selected structural analysis procedure in the framework of robust optimum seismic design as well as the design for minimum life-cycle cost and environmental impact.

Last but not least, it is always important, in real-life applications, that constraints related to the compatibility of the structural solutions with architectural requirements are also taken into account in the optimization procedure.

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