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Life-cycle cost optimization of steel moment-frame structures: performance-based seismic design approach

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Abstract. In recent years, along with the advances made in performance-based design optimization, the need for fast calculation of response parameters in dynamic analysis procedures has become an important issue. The main problem in this field is the extremely high computational demand of time-history analyses which may convert the solution algorithm to illogical ones. Two simplifying strategies have shown to be very effective in tackling this problem; first, simplified nonlinear modeling investigating minimum level of structural modeling sophistication, second, wavelet analysis of earthquake records decreasing the number of acceleration points involved in time-history loading. In this paper, we try to develop an efficient framework, using both strategies, to solve the performance-based multi-objective optimal design problem considering the initial cost and the seismic damage cost of steel moment-frame structures. The non-dominated sorting genetic algorithm (NSGA-II) is employed as the optimization algorithm to search the Pareto optimal solutions. The constraints of the optimization problem are considered in accordance with Federal Emergency Management Agency (FEMA) recommended design specifications. The results from numerical application of the proposed framework demonstrate the capabilities of the framework in solving the present multi-objective optimization problem.

Keywords: performance-based design; nonlinear dynamic analysis; steel moment-frame structure; life-cycle cost; non-dominated sorting genetic algorithm; simplified nonlinear modeling; wavelet analysis

1. Introduction

The total cost of a structure is not only dependent on the initial construction costs but also the secondary costs such as maintenance, damage, and repair expenses have a great impact on the entire expected cost of the structure in its lifetime and should be included in the decision-making process (Kaveh *et al*. 2011). In the literature, the entire expected life cost of a structure is known as life-cycle cost (Liu *et al*. 2003). It is shown that an optimum design with respect to the minimum initial cost is far from being optimum with respect to the total cost corresponding to the lifetime of a structure. Hence, an optimized seismic design is obtained when it can achieve balanced minimization of two general conflicting objective functions: the present capital investment and the future seismic risk (Liu *et al*. 2005).

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Performance-based engineering is an emerging philosophy for design, rehabilitation and maintenance of new or existing engineering structures. This approach aims at overcoming the limits of current design codes that are based on deterministic structural analyses and prescriptive procedures intended to preserve life safety (Foley *et al*. 2007). The most distinctive feature of the new approach compared to the conventional design codes is the explicit requirement of deformation-based structural performance under different hazard levels in order to achieve structural designs that not only reliably protect human lives after rare ground motions, but decrease damage after more frequent ground motions. The damage state associated with each hazard level is defined by deformation indices as a measure of distortion severity that structures will experience during significant earthquake events of that particular level. Thus, the concept of damage control can be incorporated in the design stage for reducing future economic losses, rather than just design structures for severe damage states as required in the conventional design codes (Liu *et al*. 2003).

Interstory drift ratio, defined as the difference in lateral displacements in between two consecutive floors normalized by the interstory height, is the response parameter recommended by FEMA-350 (2000) for judging the ability of a structure to resist the P-Δ instability and collapse. It is also closely related to plastic rotation demand, or drift ratio demand, on individual beam-column connection assemblies, and is therefore a good predictor of the performance of beams, columns and connections .Wen and Kang (2001) have proposed a method using the exceedance probability of the maximum interstory drift ratio from predefined drift levels for predicting the seismic damage cost of structures. This method has been successfully adopted in several researches in the last decade (e.g., Liu *et al*. 2003; Fragiadakis *et al*. 2006; Kaveh *et al*. 2011). In this method, a structural analysis procedure is needed to be performed in order to predict the values of maximum interstory drift ratios at different seismic hazard levels. The ability to reliably estimate the probable performance of a structure is dependent on the ability of the analysis procedure to predict the values of response parameters within acceptable levels of confidence. The linear procedures are the most unreliable approaches due to inherently so many uncertainties with their estimates of the structural deformation capacity. The nonlinear static procedure is more reliable than the linear procedures in predicting response parameters for structures that exhibit significant nonlinear behavior, particularly if they are irregular. However, it does not accurately account for the effects of higher mode response. If appropriate modeling is performed, the nonlinear dynamic approach is most capable of capturing the probable behavior of the real structure in response to ground motion, since all modes of vibration, geometric and material nonlinearities, and second-order effects can be directly included in the analysis (FEMA-350 2000).

Nevertheless, extensive computational demand has limited the widespread application of nonlinear dynamic analyses in practice. This problem is intensified when dynamic analyses are applied to iterative procedures such as optimization. Optimization algorithms usually need to perform a large number of fitness function evaluations in order to obtain a good solution. Our optimization problem requires even more complicated evaluations, because each fitness function evaluation contains two time-history analyses that takes a considerable time even if advanced computers are employed.

Performance-based design optimization is a combination of performance-based seismic engineering and meta-heuristic algorithms into an automated design environment where design optimization is implicitly built into the process (Foley *et al*. 2007). In recent years, several studies have been conducted on this subject. Ganzerli *et al*. (2000) minimized overall material cost for a simple reinforced concrete portal frame with performance constraints on beam and column plastic rotations. Foley (2002) discussed the application of structural optimization techniques in a performance-based design framework. Liu *et al*. (2005) formulated the performance-based seismic design of steel structures as a multi-objective optimization problem, in which conflicting design criteria that respectively reflect the primary investment and the seismic repair cost were considered. Fragiadakis *et al*. (2006) proposed a methodology for the performance-based optimal design of steel structures using static pushover analysis in order to determine the level of damage for different earthquake intensities. Alimoradi *et al*. (2007) provided a multi-objective optimization procedure for design of steel frames based on the probabilistic performance-based formulations, employing nonlinear dynamic analysis as the analytical basis. Kaveh *et al*. (2011) created a framework for the optimum seismic design of steel structures based on life-cycle cost considerations with the purpose of decreasing the computational burden of required pushover analyses during the optimization process to make the procedure feasible for large-scale structures. Karami Mohammadi and Sharghi (2014) presented a practical method for the performance-based optimal design of eccentrically braced steel frames based on the concept of uniform deformation theory.

The aim of this study is to develop a practical and automated framework for the optimum performance-based design of steel moment-frame structures with an acceptable computational time. As mentioned above, structural optimization using nonlinear dynamic analysis procedures is highly computation intensive. In this study, we try to incorporate the available techniques in the literature into a simple framework in order to make the solution of our problem possible in a timely manner.

Minimization of the life-cycle cost is considered by treating the initial and the seismic damage cost as two separate objectives of the optimization problem. The meta-heuristic employed in this study belongs to a subclass of evolutionary algorithms. NSGA-II (Deb *et al*. 2002) is a well-known, fast sorting and elitist multi-objective genetic algorithm. The wide application of this algorithm in engineering problems proves its great abilities in covering the Pareto front and solving multi-objective optimization problems (Deb 2009, Talbi 2009).

During recent years, several studies have been carried out on finding methods to reduce the computational burden of time-history analyses. Some of these studies are focused on developing simplified structural models of steel moment-frames with fewer degrees of freedom compared to models with member-by-member representation in order to quickly predict earthquake responses of structures. Nakashima *et al*. (2001) developed a generic frame model for the simulation of earthquake responses of steel moment-frames in which all beams at each floor level are condensed into one rotational spring, and all columns in each story are condensed into one representative column. In this model, overturning moment and axial deformations in columns are neglected. Lignos *et al*. (2011) successfully developed and tested a simplified nonlinear model of steel moment-frames against time-history loadings for different demand parameters such as interstory drift ratios, story shear forces, and absolute floor accelerations. In this model, a single bay frame represents the original multi-bay moment-frame so that overturning moment and column axial deformation effects are adequately represented. The experimental results demonstrated that the proposed simplification in modeling, to a great extent, maintains accuracy in predicting the desired response parameters. Simplified modeling seems to be valuable in performance-based design optimization, where response parameters need to be computed many times during the optimization algorithm. By using these models, the computation time for optimization procedure can reduce dramatically, because they require the solution of significantly fewer degrees of freedom in comparison with member-by-member frame models.

Some studies focus on producing surrogate records for original earthquake records that have larger time steps but with almost similar effects on structures. In this field, wavelet analysis has shown to be very effective. Wavelet decomposition can divide an earthquake signal into two parts: low frequency approximation part and high frequency detail part. Low frequency part is the most influential part of the original signal on the response of structures and it can efficiently be used in dynamic analysis of structures to decrease the number of points of earthquake record involved in the time-history loading (see Salajegheh *et al*. 2005; Gholizadeh *et al*. 2011; Kaveh *et al*. 2012). In this paper, both introduced strategies (simplified structural modeling and wavelet analysis) will be implemented in the proposed framework for reducing computational time of the optimization procedure.

The remaining parts of the paper are organized as follows: section 2 explains the concept of performance-based design with details; section 3 briefly introduces the NSGA-II; section 4 explains the calculation of the seismic damage cost; section 5 defines the details of the simplified nonlinear modeling; section 6 illuminates the main ideas behind using wavelet analysis; section 7 presents the proposed framework; Section 8 studies the performance-based optimal design of a ten-story steel moment-frame structure and finally the paper is concluded with Section 9.

2. Performance-based design procedure

Minimization of life-cycle cost for an individual structure can be achieved by optimizing its performance at different seismic hazard levels. FEMA-350 (2000), *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*, evaluates structural performance at two levels of seismic hazard:

- Maximum considered earthquake (MCE) ground motions with less than 2% probability of exceedance in 50 years;
- Frequent earthquake (FE) ground motions with 50% probability of exceedance in 50 years.

Under FEMA-350, each building and structure must be assigned to one of three Seismic Use Groups (SUGs). Buildings are assigned to the SUGs based on their intended occupancy and use. Most commercial, residential and industrial structures such as those studied in this paper are assigned to SUG I. FEMA-350 states that all buildings should, as a minimum, be designed in accordance with the applicable provisions of the prevailing building code, i.e. AISC-LRFD (2010) specifications; in case the building is to attain a performance other than what the building code implies, the performance evaluation procedure may be followed according to FEMA-350. In the two-step procedure of FEMA-350 for performance evaluation, at each step one performance objective is verified. Each performance objective consists of the specification of a structural performance level and a corresponding hazard level, for which that performance level is to be achieved. Performance objectives for SUG-I structures are as follows:

- Collapse prevention building performance level for earthquake demands that are less severe than the MCE ground motions;
- Immediate occupancy building performance level for earthquake demands that are less severe than the FE ground motions.

Buildings that achieve immediate occupancy (IO) level are expected to sustain minimal or no damage to their structural elements; and only minor damage to their nonstructural components, so immediate re-occupancy of the building is safe. At collapse prevention (CP) level, buildings may pose a significant hazard to life safety resulting from failure of nonstructural components. However, since the building does not collapse, gross loss of life could well be avoided. Many buildings that achieve this level are complete economic losses (FEMA-350 2000). Although nonstructural components damage is extremely important, the present methods for the estimation of potential seismic damage only consider structural components.

In order to evaluate the performance of a structure through the mentioned seismic hazard levels, it is necessary to construct a mathematical model of the structure that can represent its strength and deformation characteristics, and then to conduct a nonlinear dynamic analysis to predict the values of various demand parameters at each hazard level. In a nonlinear dynamic analysis, the response of a structure to a ground motion time-history is determined through numerical integration of the equation of motion for the structure. Structural stiffness is altered during the analysis to conform to the nonlinear hysteretic models of the structural components. In this study, we utilized the advanced capabilities of OpenSees® (2013) in modeling and analyzing the nonlinear response of structures. OpenSees® is an open-source software for simulating the seismic response of structural and geotechnical systems. This software was developed to serve as a computational platform for research in performance-based earthquake engineering at the Pacific Earthquake Engineering Research Center.

2.1 Ground motion characterization

FEMA-350 states that the ground motion acceleration histories should be prepared in accordance with the recommendations of FEMA-273 (1997). For 2D structures, the analysis should be performed with a suite of not less than three ground motion time-histories, each containing a horizontal component. Time histories should have magnitude, fault distances, and source mechanisms that are equivalent to those that control the design earthquake ground motion. Where three recorded ground motion time-history data sets having these characteristics are not available, simulated time-history data sets having equivalent duration and spectral content could be used to make up the total number required. The acceleration time-histories should be scaled such that the average value of the 5%-damped response spectra for the suite of motions does not fall below the target response spectrum for the site for periods between 0.2T seconds and 1.5T seconds, where T is the fundamental period of the structure (FEMA-273 1997).

The performance evaluation procedure of FEMA-350 consists of two analysis steps, each associated with one of the two seismic hazard levels. At each step, the analysis should be performed for a suite of ground motion records that have been scaled to the respective target response spectrum. FEMA-273 offers equations for calculating the response spectrum of MCE ground motions in which the required seismic input data can be found on the ground-shaking hazard maps provided by this document. Based on FEMA-273, the FE response spectrum for California is 0.29 of the shaking intensity for the MCE spectrum at each period.

In this study, only one ground motion is used at each hazard level to reduce computational demand. Moreover, instead of using a real ground motion time-history -in order to have a more appropriate record compatible with the seismic characterization of the site- the real earthquake record is adjusted and scaled using SeismoArtif® (2012) to generate an artificial earthquake record matched to the target response spectrum. SeismoArtif® is a software capable of generating artificial earthquake records matched to a specific target response spectrum using different calculation methods and varied assumptions. In cases in which real records are -for any reason- incompatible to process, a tool such as SeismoArtif® is of pertinence and usefulness.

Since only the maximum response of structure is needed for the performance evaluation, in order to reduce the computational burden, the effective duration of the ground motion can be used in the analysis instead of considering the whole earthquake record. The effective duration of a ground motion determines the start and the end of a strong shaking phase that is the time interval between the accumulation of 5% and 95% of ground motion energy, where ground motion energy is defined by the Arias intensity (Towhata 2008). The end of the duration is the time until which the maximum response has already been recorded. Therefore, in order to achieve a time efficient analysis, the record is to be analyzed up to the end of the effective duration and further analysis is not necessary (Kaveh *et al*. 2012). The effective duration of an earthquake record can be easily computed by software such as SeismoSignal® (2002). In this study, the concept of effective duration is applied to shorten the duration of the given earthquake record, however, the start of the duration is considered to be from the start of the given earthquake record. Moreover, in order to further decrease the number of points of the earthquake record involved in the time-history loading, a wavelet decomposition procedure is applied. Details of this procedure are provided in Section 6.

2.2 General requirements and performance evaluation

The seismic provisions of FEMA-350 for the design of new steel moment-frame structures state in order to check the validity of any design alternative, firstly, the required strength of structural members and connections must be verified by AISC-LRDF specifications. Strength checks can be found in any textbook about the design of steel structures. In this study, the equivalent lateral force procedure of ASCE-7 (2010) is used for seismic design of structures. In the next stage, structures should be analyzed under FE and MCE ground motions to check whether the acceptance criteria at the IO and CP performance levels are met.

FEMA-350 presents a probabilistic procedure that evaluates structural performance in terms of confidence levels for specified structural response parameters, including, interstory drift ratio, column axial compression force and column (splice) tension force. These structural response parameters are related to the amount of damage experienced by individual structural components as well as the structure as a whole. For each performance level, FEMA-350 specifies acceptance criteria (median estimates of capacity) for each of these response parameters. Acceptance criteria have been developed on a reliability basis, incorporating demand and resistance factors related to the uncertainty inherent in the evaluation process and variation inherent in structural response, so that a confidence level can be estimated with regard to the ability of a structure to meet the desired performance objectives. If an evaluation indicates a high level of confidence -for example 90 or 95%- it is very likely (but not guaranteed) that the building will be capable of meeting the desired performance. If a lower confidence is calculated, -for example 50%- this is an indication that the building may not be capable of meeting the desired performance objective. If still a lower confidence is calculated -for example 30%- the building is likely not to meet the desired performance objective (FEMA-350 2000).

Although column axial compression and tension, and connection drift ratio are important response measures in assessing performance of steel frames, due to the limitations of the current study, they were not included in our calculations. If one assumes global behavior is limited by interstory drift ratio as the controlling response parameter -as done in the present work- the FEMA-350 methodology requires at least 50% confidence in attaining IO performance objective and 90% confidence in achieving CP performance objective. Consequently, in this study for the mid-rise special steel moment-frames, the maximum allowable interstory drift ratio of 1.5% and 5% are considered at the IO and CP performance levels, respectively.

3. Optimization algorithm: NSGA-II

The NSGA-II algorithm and its detailed implementation procedure can be found in (Deb *et al*. 2002). In the following, a general description of NSGA-II is provided. Once the population is initialized, two fitness values are assigned to each individual. Firstly, NSGA-II uses a "non-dominated sorting" algorithm for fitness assignment in which all individuals that are not dominated by any other individual are assigned front number 1; all individuals only dominated by the individuals in front number 1 are assigned front number 2, and so on. Secondly, a value called "crowding distance" is calculated for each individual; it is a measure of how close an individual is to its neighbors. A higher fitness value is assigned to individuals located on the sparsely populated part of a front (Deb 2009).

Parent selection is made using a "binary tournament selection" based on the assigned fitness values. This selects, between two random individuals, the one with the lowest front number, if the two individuals are from different fronts. While the individuals are from the same front, the individual with the highest crowding distance is selected. Then, the selected individuals generate offsprings using genetic operators. The offspring population is combined with the current generation's population, replacement is performed to set the individuals of the next generation. Since all previous and current best individuals are included, elitism is ensured. The combined population is now sorted based on the non-domination rule. The new generation is filled with fronts, one after another, until the population size exceeds the given size. If by adding all the individuals from the *i*th front, the population size exceeds, then individuals in the *i*th front are selected based on their crowding distance in a descending order until the population is formed. This process is repeated to generate the subsequent generations, until the termination criteria is met (Deb 2009).

3.1 Genetic operators

In this study, the chosen genetic operators are differential evolution (DE) operator for crossover and polynomial mutation operator. The role of crossover operator is to inherit some genetic materials of parents to generate offsprings, whereas mutation alters one or more gene values in a chromosome from its initial state. The mutation and crossover operators are complementary, i.e. mutation maintains genetic diversity from one generation of a population of algorithm chromosomes to the next while crossover preserves genetic inheritance between generations (Talbi 2009).

Unlike the classical crossover operators of genetic algorithms where parts of parents are recombined, the DE operator is based on a linear combination of individuals. The algorithm of the DE operator can be described as follows (Talbi 2009):

DE Crossover operator {

1. Select randomly a parent x and three other different individuals r^1, r^2 and r^3 from parents population.
2. Pick a random index $i_{rand} \in \{1,...,n\}$, where n is the dimensionality of the problem.
3. Generate an offspring $x' = \{x'_1,...,x'_n\}$.
   3.1. For each gene, pick a uniformly distributed number $rand_i$ in the range [0, 1].
   3.2. If $rand_i < CR$ or $i = i_{rand}$ then set $x'_i = \text{Fix}(r_i^3 + F(r_i^1 - r_i^2))$, else set $x'_i = x_i$.

}.

The parameter $F \in [0,1]$ represents the differential weight controlling the amplification of the difference between individual r^1 and r^2, it is used to avoid stagnation of the search process. The parameter $CR \in [0,1]$ is called the crossover probability. With the probability *CR*, the offspring associated variable is a linear combination of three randomly chosen solutions; otherwise the variable inherits the value of its parent. The condition $i = i_{rand}$ is included to ensure at least one variable of the offspring is different from its parent. In this study, the values of *F* and *CR* are respectively set to 0.8 and 0.9.

After the crossover operator is performed, mutation takes place on the newly formed individual. In the polynomial mutation, offspring is generated as follows (Talbi 2009):

$$x''_i = \text{Fix}(x'_i + (u_i - l_i)\delta_i) \tag{1}$$

where u_i (resp. l_i) represents the upper bound (resp. lower bound) for x'_i, the *i*th variable (gene) of the parent. The parameter δ_i is computed from the polynomial probability distribution:

$$\delta_i = \begin{cases} (2rand_i)^{\frac{1}{\eta_m+1}} - 1 & \text{if } rand_i < 0.5 \\ 1 - (2(1 - rand_i))^{\frac{1}{\eta_m+1}} & \text{otherwise} \end{cases} \tag{2}$$

where η_m is the distribution index and $rand_i$ is a random number in [0, 1]. Mutation should produce a minimal change and the size of mutation should be controllable. The parameter η_m provides these features, e.g. taking η_m to be 20, limits the values of δ_i in [-0.4, 0.4]. During crossover or mutation, if a variable of the new individual goes out of the boundary of the acceptable domain, its value is reset to the value on the nearest boundary.

3.2 Constraints handling

In order to handle the given constraints, a relatively simple scheme is adopted. Whenever two individuals are compared for sorting population in different fronts, first, they are checked for constraint violation. If both are feasible, the non-domination rule is directly applied to decide the winner. If one is feasible and the other is infeasible, the feasible dominates. If both are infeasible, the one with the lowest amount of constraint violation dominates the other. This is the approach that was utilized by (Deb *et al*. 2002; Coello *et al*. 2004) to handle the constraints.

4. Lifetime seismic damage cost

For a structure that is designed against probable earthquakes -ignoring the maintenance cost- the whole expected life cost of the structure is the total of the initial cost and the seismic damage cost. The initial cost of a new structure refers to the cost of the structure during the construction stage. For steel-framed structures, initial cost is usually considered to be proportional to the total weight of the structural components (Fragiadakis *et al*. 2006). Lifetime seismic damage cost, in general, refers to the potential damage cost from earthquakes that may occur during the life of a structure including the cost of damage after an earthquake, loss of contents, injuries or human fatalities, and other direct or indirect economic losses (Liu *et al*. 2003). However, in the field of structural engineering, the main focus is on estimating the cost of structural damage. Within a performance-based design framework, seismic damage cost can be related to different levels of damage state violations (Liu *et al*. 2003). In this study, similar to Wen and Kang (2001), seven damage states in terms of maximum interstory drift ratios -see Table1- are used to describe the respective performance levels. Based on a Poisson process model of earthquake occurrences and an assumption that damaged buildings are retrofitted to their original intact conditions after each major seismic attack, Wen and Kang (2001) proposed the following formula for calculation of the expected lifetime seismic damage cost of a structure:

$$C_{LSD} = \frac{\nu}{\lambda}(1 - e^{-\lambda t})\sum_{i=1}^{n} C_i P_i \tag{3}$$

where n is the total number of damage states considered; t is the service lifetime of a new structure or the remaining lifetime of a retrofitted structure; ν is the annual occurrence rate of significant earthquakes; λ is the annual momentary discount rate assumed to be constant and equal to 5%; C_i is the retrofitting cost of ith damage state violation, expressed as a percentage of the initial cost (see Table 1, column 4); and P_i is the probability of ith damage state violation, calculated by:

$$P_i = P_i(\Delta > \Delta_i) - P_i(\Delta > \Delta_{i+1}) \tag{4}$$

in which Δ_i and Δ_{i+1} are the drift ratios defining the lower and the upper bound of ith damage state; $P_i(\Delta > \Delta_i) = -1/(\nu.t)\ln(1 - P_t(\Delta > \Delta_i)))$, where $P_t(\Delta > \Delta_i)$ is the exceedance probability over a period (0, t). Note that ν can be cancelled in the above equations if the retrofitting cost associated with the first damage state, C_1, is zero (Liu *et al*. 2005). This is the case considered in this study.
The damage states listed in Table 1 are reached when the respective maximum interstory drift ratios exceed from the limits noted in column 3 of this table. In order to calculate $P_i(\Delta > \Delta_i)$, one should first calculate the annual exceedance probability of the ith damage state, i.e. $P_{t=1}(\Delta > \Delta_i)$,

Table 1 Structural performance levels and damage states in terms of interstory drift ratios, and corresponding retrofitting costs (from Wen and Kang 2001)

Performance level	Damage state	Interstory drift ratio (%)	Cost (% of initial cost)
I	None	Δ<0.2	0
II	Slight	0.2<Δ<0.5	0.5
III	Light	0.5<Δ<0.7	5
IV	Moderate	0.7<Δ<1.5	20
V	Heavy	1.5<Δ<2.5	45
VI	Major	2.5<Δ<5.0	80
VII	Destroyed	5.0<Δ	100

which can be derived from the following relationship based on the work of Fragiadakis *et al*. (2006):

$$P_{t=1}(\Delta > \Delta_i) = \alpha e^{-\beta \Delta_i} \tag{5}$$

where the parameters α and β are obtained by the best fit of the known pairs of $P_{t=1}$ and Δ_i. In accordance with FEMA-350, these pairs may correspond to the annual probabilities of exceedance of FE and MCE ground motions that have respectively 50% and 2% probability of exceedance in 50 years; and the respective maximum inter-story drift ratios, Δ_i, can be obtained from the time-history analyses. According to Poisson's law, the annual probability of exceedance of an earthquake with a probability of exceedance p in t years is given by $\overline{P} = -1/t.\ln(1-p)$ (Fragiadakis *et al*. 2006). For calculation of $P_i(\Delta > \Delta_i)$ in Eq. (4), the annual exceedance probability of the drift ratio defining the *i*th damage state, $P_{t=1}(\Delta > \Delta_i)$, is directly read from the fitted curve.

5. Simplified nonlinear modeling

Simplified modeling investigates the minimum level of multi-degree-of-freedom modeling sophistication that results in a negligible loss of accuracy in predicting demand parameters. This approach has shown to be highly effective in reducing the computational effort for estimating seismic demands of steel moment-frame structures (see Nakashima et al. 2001). In the method developed by Lignos *et al*. (2011), as shown in Fig. 1, a multi-bay steel moment-frame is condensed to a single bay simplified frame with properties tuned to represent the original frame. Lumping together a multi-bay frame into a single bay frame can be accomplished by the following rules:

$$\sum EI_i / L_i = EI/L \tag{6}$$

$$\sum M_{p,i} = M_p \tag{7}$$

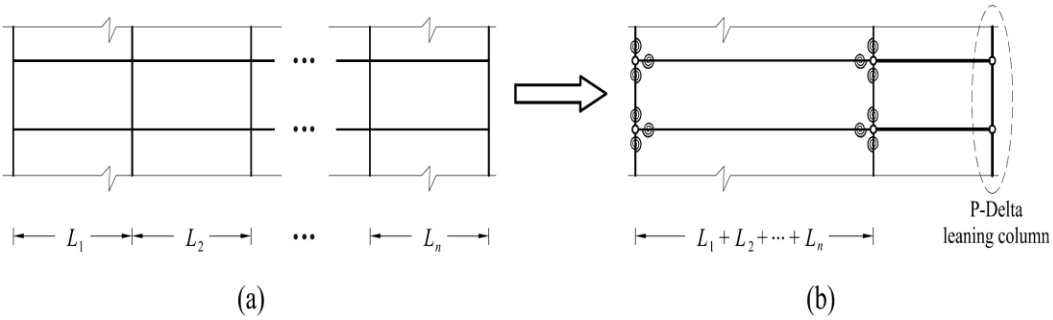

Fig. 1 Simplified nonlinear modeling: (a) member-by-member model of an n-bay steel moment-frame in a story; (b) simplified nonlinear model of the frame in that story

where I_i and L_i is the moment of inertia and the length of the *i*-th beam in a story, respectively, and EI/L and M_p are the stiffness and the plastic moment of the single bay beam. For steel columns:

$$\sum EI_i = 2EI \tag{8}$$

$$\sum M_{pc,i} = 2M_{pc} \tag{9}$$

in which $M_{pc,i}$ is the plastic moment of the *i*-th column of the multi-bay frame and M_{pc} is the plastic moment of the single bay columns in presence of axial load. For higher steel moment-frames in which overturning moment and axial deformations in columns are important, these effects can be included by setting L of the single bay frame equal to the distance between end columns of the multi-bay frame, and setting the cross-sectional area of the single bay columns equal to the area of the end columns of the multi-bay frame. This simplification is based on the assumption that overturning effects are resisted mostly by the exterior columns of a steel moment-frame (Lignos *et al*. 2011). In the simplified model, a leaning column carrying gravity loads is linked to the frame by axially rigid truss elements, to simulate P-Delta effects of the existing gravity columns on the response of the lateral resisting frame. The approximations considered in this method are reasonable if all bays of the frame are of about equal width, and they are more approximate when spans of the frame vary considerably (Lignos *et al*. 2011).

The simplified model, as shown in Fig. 1(b), consists of elastic beam-column elements with rotational springs at their ends. The rotational springs capture the nonlinear behavior of the frame consistent with the concentrated plasticity concept. The rotational behavior of the plastic hinges follows a bilinear hysteretic response based on the modified Ibarra-Krawinkler deterioration model. Detailed information about this model and the modes of deterioration it simulates are available in (Ibarra and Krawinkler 2005; Lignos and Krawinkler 2011). In this study, the input parameters for the rotational behavior of the plastic hinges are determined using the empirical relationships developed by Lignos and Krawinkler (2011), derived from an extensive database of steel component tests. For the sake of simplicity, in this paper, cyclic deterioration is ignored in studying the nonlinear behavior of structures.

Since frame member is modeled as an elastic element connected in series with rotational springs at either end, the stiffness of these components must be modified so that the equivalent stiffness of this assembly is equivalent to the stiffness of the actual frame member. Using the approach described in appendix B of (Ibarra and Krawinkler 2005), the rotational springs are made n times stiffer than the rotational stiffness of the elastic element in order to avoid numerical problems and allow all damping to be assigned to the elastic element. In order to ensure that the equivalent stiffness of the assembly is equal to the stiffness of the actual frame member, the stiffness of the elastic element must be $(n+1)/n$ times greater than the stiffness of the actual frame member. In this study, this is accomplished by making the elastic element's moment of inertia $(n+1)/n$ times greater than the actual frame member's moment of inertia. Ibarra and Krawinkler (2005) suggested a value of $n=10$ for the stiffness modifications.

Moreover, to match the nonlinear behavior of the assembly with that of the actual frame member, the strain hardening coefficient (the ratio of post-yield stiffness to elastic stiffness) of the plastic hinge must be modified, as well. If the strain hardening coefficient of the actual frame member is denoted by $\alpha_{s,mem}$, and the strain hardening coefficient of the spring by $\alpha_{s,spring}$, then $\alpha_{s,spring} = \alpha_{s,mem} / (1+n(1-\alpha_{s,mem}))$. More information about the modified Ibarra-Krawinkler deterioration model, its features and implementation, is available on the online supporting documentation provided by OpenSees® developers.

In order to test the accuracy of the simplified nonlinear models in estimating the required response parameters, the ten-story moment-frame introduced in the numerical study section was modeled with both the simplified and the member-by-member representation for 100 randomly generated design variables and analyzed against the same earthquake record. The mean value of errors -evaluated by the RRMSE (relative root mean squared error) measure- in predicting inter-story drift ratios was 0.0016. The total computation time recorded for analyzing the member-by-member models was 25.6 min and for the simplified models was 4.7 min. These values demonstrate that the simplified models can reproduce interstory drift ratios with considerable accuracy, consuming much fewer time, as well. These features make the simplified nonlinear modeling a reliable and effective tool for improving time efficiency of the nonlinear dynamic analysis procedures.

6. Wavelet analysis

Wavelet analysis is an advanced mathematical set of tools and techniques for signal-processing which has aroused great attention in many fields of science and engineering. By a wavelet decomposition, we can denoise a signal from high-frequency components to understand behavior of the primary signal better. The theory and methods of wavelet analysis are widely available in literature. In this paper, only the application of wavelet analysis in our problem is explained; additional information can be found in (Strang *et al*. 1996).

Wavelet transform is a method for decomposing data, functions and signals into different frequency bands (Salajegheh *et al*. 2005). A wavelet transform can be simply constructed by a tree of filter banks as shown in Fig. 2. In this figure, "downsampling" is an operation that keeps the even indexed elements of the input signal. The key scheme for a wavelet transform is to

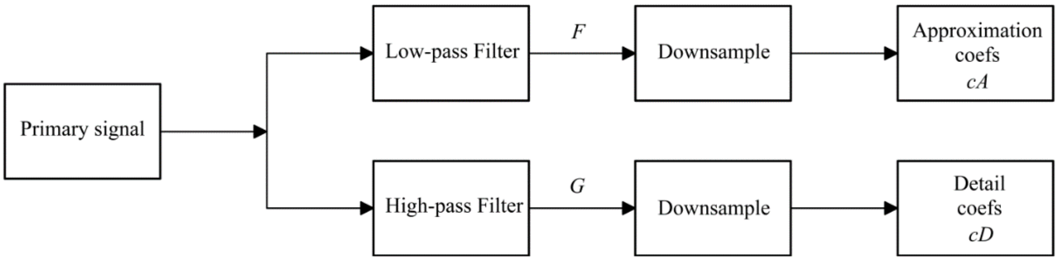

Fig. 2 General algorithm for discrete wavelet transforms

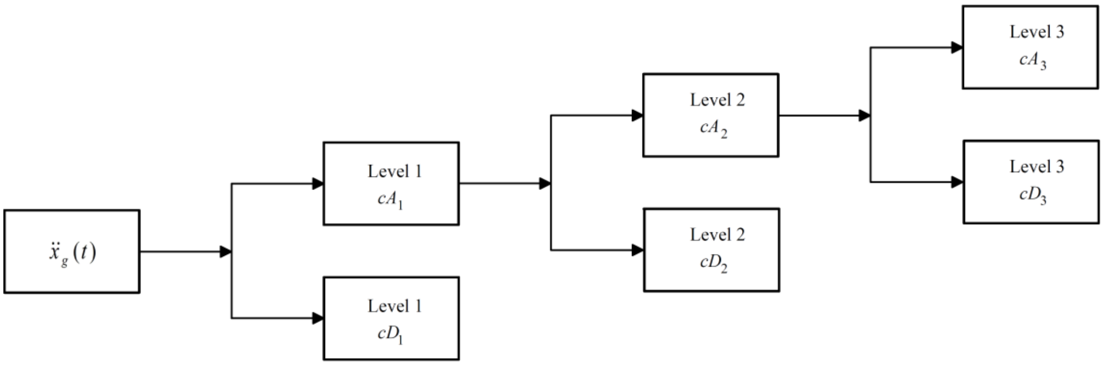

Fig. 3 A three-level wavelet decomposition of earthquake record $\ddot{x}_g(t)$

decompose a signal into two parts: the low-frequency part and the high-frequency part. This scheme is achieved by a set of filters (a low- and a high-pass filter), which separate the input signal into different frequency bands. The low-pass filter removes the high-frequency bands of the input signal and produces an approximate signal; the high-pass filter removes the low-frequency bands and produces a signal including the details of the input signal (Kaveh *et al*. 2012). In other words, by constructing a wavelet transform with these two filters, the input signal is decomposed into an approximation and a detail signal. As shown in Fig. 2, the output of a wavelet transform is two sets of coefficients, (*cA*) and (*cD*), respectively include the low- and the high-frequency content of the input signal. In Fig. 2, the length of each filter is equal to 2*N* that N is the order of the wavelet function used for the filter. If *n* is the length of the input signal, the signals *F* and *G* are both of length $n + 2N - 1$; and the length of *cA* and *cD* are equal to floor $\left(\frac{n-1}{2}\right)+N$, almost half of the input signal length.

The dynamic response of structures is mostly affected by the low-frequency content of the earthquake records (Salajegheh *et al*. 2005). This content can be efficiently used in dynamic analysis of structures, as a surrogate record for the given earthquake record, in order to decrease the number of acceleration points involved in the time-history loading and subsequently reduce the computational demand of this type of analysis. The decomposition process can be repeated for the low-frequency content to achieve the desired scale of the earthquake record. This multilevel decomposition process is called wavelet decomposition tree (Gholizadeh *et al*. 2011). In this study, the decomposition process proceeds in three levels (see Fig. 3), i.e. the approximate version of the earthquake record in the last step (cA_3) is used for the dynamic analysis of structures. Therefore, the number of points involved in the time-history loading is decreased to 0.125 of the given earthquake record. When cA_3 is applied, the time-step of dynamic analysis must be updated by (Kaveh *et al*. 2012):

$$dt = dt \times \frac{length(\ddot{x}_g)}{length(cA_3)} \tag{10}$$

In the present problem, wavelet decomposition process is needed to be performed once before the start of the optimization procedure. In the phase of preparing the input data, the decomposition tree produces a surrogate record for the given earthquake record, which is used instead of it in all dynamic analyses of the procedure. According to the results of our previous study (Kaveh *et al*. 2012), Daubechies wavelet function (Db2) is selected to operate as the filter and decompose the earthquake record. In order to verify the efficiency of the three-level wavelet decomposition tree, 100 simplified models for the ten-story moment-frame introduced in the numerical study section were generated randomly and analyzed subjected to both the Loma Prieta earthquake record and its surrogate record. The mean value of the errors in estimating interstory drift ratios using the surrogate record was 0.0328, calculated by the RRMSE measure. While the analysis time for this record was approximately 1/8 of that was required for the original record. These values confirm that by implementing the developed wavelet decomposition method, considerable improvement in computational effort is achieved at the expense of a small loss of accuracy.

7. The proposed framework

Now, all of the introduced components in the previous sections are incorporated in a simple framework which makes it possible to solve our optimization problem. In this problem, all the constraints are classified into two main groups:

- *Initial constraints*: The constraints of this group are fulfilled by modifying the given solution. These constraints are as follows: (1) column-beam moment ratio should be satisfied at beam-to-column connections in accordance with AISC seismic provisions (2010). This condition is checked at each joint and if it is not fulfilled, the section number of the columns connecting to the joint is increased one number and then it is checked again. This process continues until all joints fulfill this constraint. (2) Lower columns should have the same or larger section number than the upper columns. This constraint is checked from the last story and gradually modifies the section of columns in order to satisfy this constraint. (3) The design strength of beams and columns should be checked following AISC-LRFD (2010) specifications. If the strength ratio of each member of structure is more than one, its section number is increased by one and this process continues until all members fulfill this constraint. The equivalent lateral force procedure of ASCE-7 (2010) is considered for earthquake loading. According to ASCE-7, the seismic load combination is $1.2D + 1.0L + 1.0E$, where D and L represent dead load and transient live load, and

E represents earthquake load.

- *Final constraints*: This group contains checking of the requirements specified in Sect. 2.2 for performance evaluation. Based on FEMA-350 (2000), the following load combination is used to evaluate seismic demands, $1.0D + 0.25L + 1.0E^{2/50}(E^{50/50})$, where $E^{2/50}$ and $E^{50/50}$ represent earthquake effects respectively under MCE and FE ground motions. For this group, constraint violation is reported by a factor that guides optimization process as mentioned in Sect. 3.2.

The main procedure, which is based on the NSGA-II algorithm, is as follows:

Main procedure {

1. *Set parameters.*
2. Initialize a population.
    2.1. Generate a random individual.
    2.2. *Evaluate* the new individual.
3. Sort the initial population based on non-domination and calculate crowding distances.
4. Select parents using binary tournament selection.
5. Generate offsprings by performing genetic operators.
    5.1. Generate a new individual.
    5.2. *Evaluate* the new individual.
6. Form an intermediate population from merging the current population with the offsprings.
7. Sort the intermediate population based on non-domination and calculate crowding distances.
8. Perform replacement on the intermediate population to determine the new population.
9. Stop if termination criterion is met, otherwise go to step 4.

}.

The first step is done as follows:

Set parameters {

1. Set the NSGA-II user defined parameters, e.g. population size, number of offsprings, number of generations, etc.
2. Select the input parameters required for structural modeling, analysis and design.
3. Generate an artificial earthquake record matched to the MCE response spectrum.
4. Define effective duration of the artificial earthquake record.
5. Perform wavelet decomposition method for the effective duration and generate a surrogate record.

}.

And, evaluation of the new individual is performed as:

Evaluate {

1. Construct a member-by-member model for the new individual.
2. Check initial constraints.
3. Construct a simplified nonlinear model for the new individual.
4. Perform two nonlinear time-history analyses respectively under FE and MCE ground motions.
5. Check final constraints.
6. Compute initial and lifetime seismic damage costs.
7. Place the new individual in the population (values of design variables, fitness values, amount of constraint violation).

}.

8. Numerical study

A computer program was developed by coding the proposed framework in MATLAB® (2011), in which structural analysis is done by the combination of MATLAB® and OpenSees®. Actually, first, the required data for analysis of structures, including structural modeling and loading, are provided by MATLAB® and then by the use of these data, OpenSees® performs the analysis. Two different models are used in this study for analyzing the given structure. In the first part, a member-by-member model of the structure is constructed using "elasticBeamColumn" element of OpenSees®; then a linear static analysis is performed to calculate design strength of the structural components under LRFD load combination. In the second part, a simplified nonlinear model of the structure is constructed with "elasticBeamColumn" elements connected by "zeroLength" elements that serve as rotational springs to represent the nonlinear behavior of the structure. Then, OpenSees® performs two nonlinear time-history analyses to estimate interstory drift ratios under FE and MCE ground motions. In order to model structural damping, Rayleigh damping model of OpenSees® is applied by assuming the damping ratio of 5% for the first and the second mode of the structure. For wavelet decomposition of the given earthquake record, the wavelet toolbox of MATLAB® is employed.

In what follows, a test problem is presented and solved using the developed program. Assume a ten-story steel frame structure with the floor plan shown in Fig. 4, in which all stories have the same plan. As observed in Fig. 4, two five-bay moment-frames in the East-West direction and five two-bay moment-frames in the North-South direction serve as the lateral load resisting system. The goal in this example is the performance-based optimal design of the moment-frame located at grid A(1-6). The member-by-member model of this frame, shown in Fig. 5(a), consists of 110 elastic beam-column elements in which all columns and beams are grouped into 13 sets, each corresponding to an independent design variable. The simplified nonlinear model of this frame, demonstrated in Fig. 5(b), consists of 30 elastic beam-column elements with rotational springs at their ends. In this model, lumping together the columns and the beams of each story is done following the rules explained in section 5. In the both models, a leaning column, carrying half of the gravity loads acting on the existing three gravity frames in the East-West direction, is linked to the frame by rigid truss elements to simulate P-Delta effects. This frame is designed as a special moment-frame based on the requirements specified in AISC seismic provisions (2010). This structure is located in Los Angeles, California, and the type of soil profile is assumed to be C at the site of the structure.

All members of the frame have I-shaped cross-sections which are selected from a database of 129 W-sections containing 23 W1000, 22 W920, 13 W840, 17 W690, 18 W530, and 36 W360 sections. Details of these standard W-sections are available in the manuals of the American Institute of Steel Construction. In order to determine the rotational behavior of the plastic hinges in the simplified model, the empirical relationships developed by Lignos and Krawinkler (2011) for I-shaped cross-sections are employed.

The modulus of elasticity is equal to 2.1e6 kg/cm^2 and the yield stress of steel is 2400 kg/cm^2.

The permanent load is considered to be D = 400 kg/m^2 and the transient live load is taken as L = 250 kg/m^2. The joint masses, for the simplified model, are computed by MATLAB® and given as input data to OpenSees®. The load combination for computing joint masses from the gravity loads is $1.0D + 0.2L$. In distributing the gravity loads, it is assumed that all loads are distributed uniformly between the two joints of each floor. In addition to the gravity loads, the self-weight

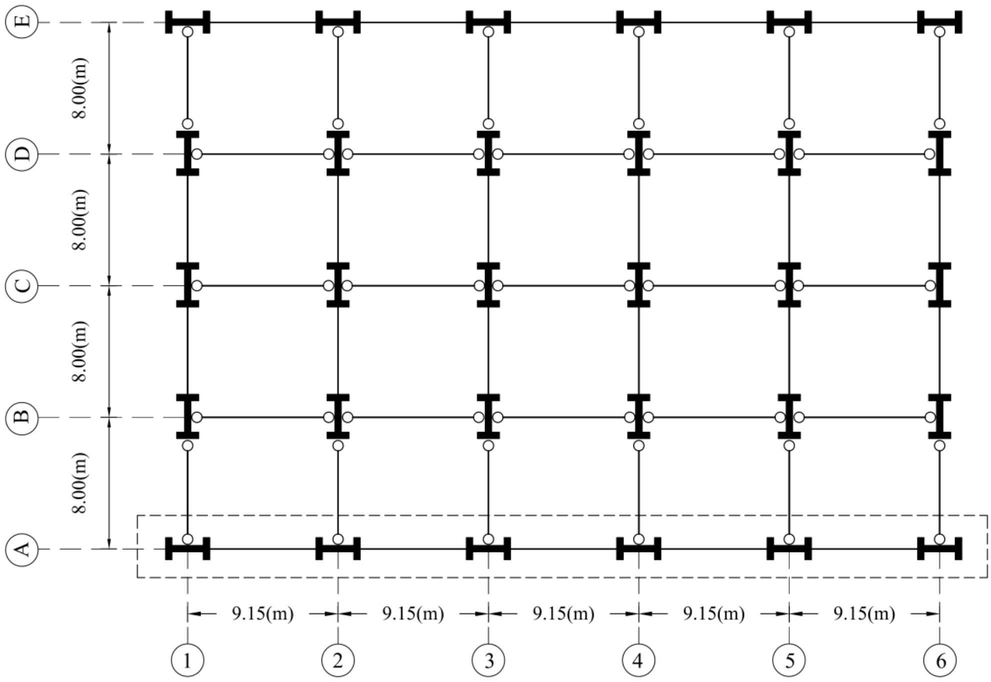

Fig. 4 Plan view of a ten-story steel moment-frame structure with the same floor plan for all stories

of each element in the member-by-member model is divided into two equal mass portions and added to the mass of the corresponding joint in the simplified model.

The Loma Prieta ground motion, see Fig. 6(a), is processed to be subjected to the frame in the horizontal direction. Details of this earthquake record is available in PEER Strong Motion database (PEER 2010). This real ground motion is adjusted and scaled using SeismoArtif® to generate an artificial record matched to the 5% damped response spectrum of MCE ground motions for the site of the structure. The generated artificial record is shown in Fig. 6(b). The effective duration of the artificial record, calculated by SeismoSignal®, stops at second 17.5 leading to 3500 points with a time step of 0.005 sec (Fig. 6(c)). The implementation of the Db2 function for the wavelet decomposition decreases the number of points to 440 with a time step of 0.0398 sec as displayed in Fig. 6(d). This filtered record is a surrogate for the artificial record that is used instead of it throughout the optimization.

Because of the stochastic nature of the solution algorithm, this problem was solved four times. The obtained Pareto fronts are shown in Fig. 7 for seismic damage cost against initial material cost (the total weight of structural components). These Pareto fronts demonstrate the rank-1 solutions obtained in the last generation of the NSGA-II algorithm for each run of the program. Since seismic damage cost is defined as a ratio of initial cost, both values are in tons. In all runs, a population of 150 individuals is evolved for 250 generations. The computational time required by the developed program to solve this multi-objective optimization problem was approximately 87 hours, using an Intel® Core™ i7 @ 2.0 GHz processor equipped with 8 GBs of RAM.

As a rough estimate, without using the employed simplified modeling, the solution algorithm requires about 473 hours, and without the three-level wavelet decomposition method,

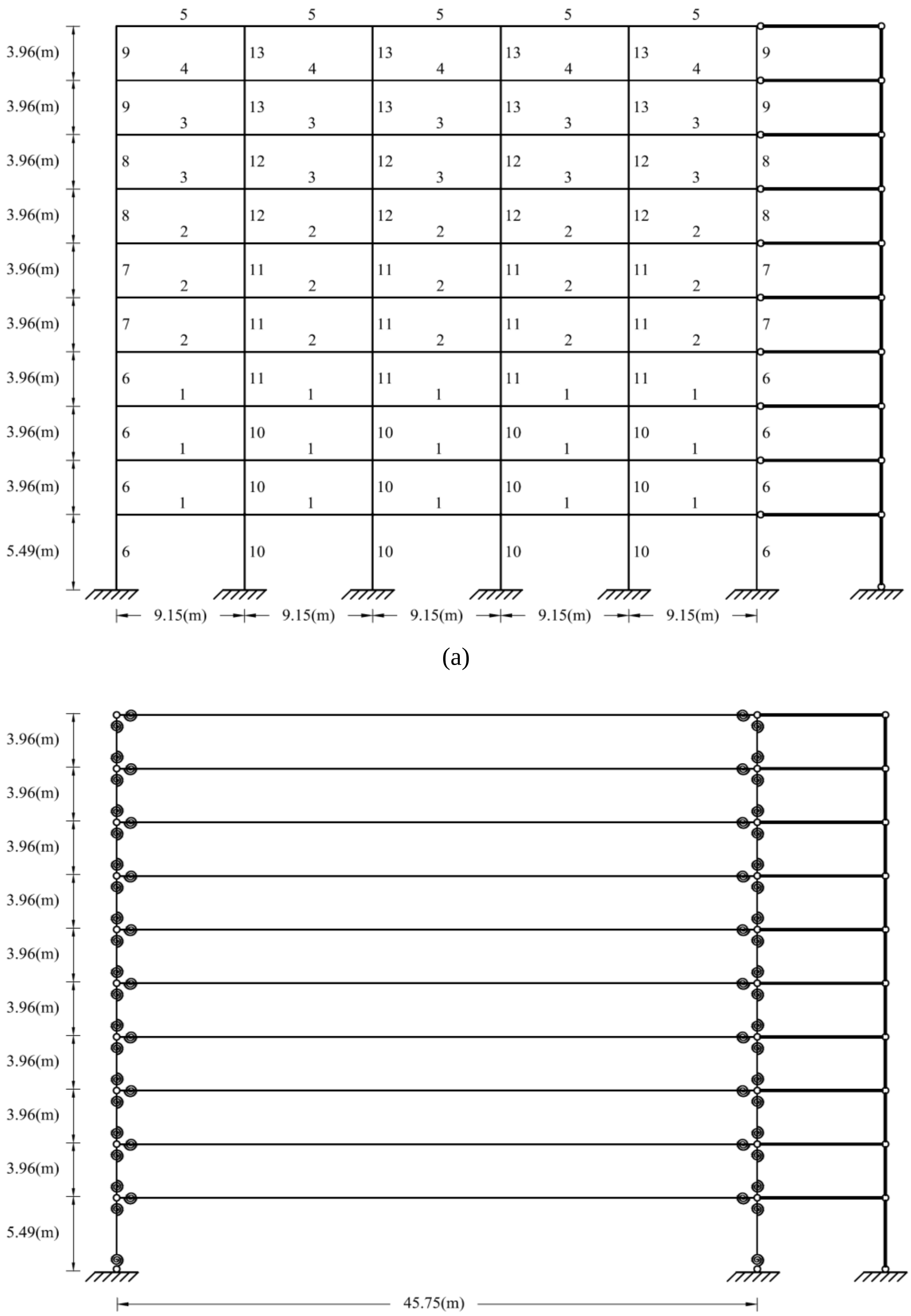

Fig. 5 Models of the moment-frame located at grid A(1-6) of the floor plan: (a) member-by-member model; (b) simplified nonlinear model

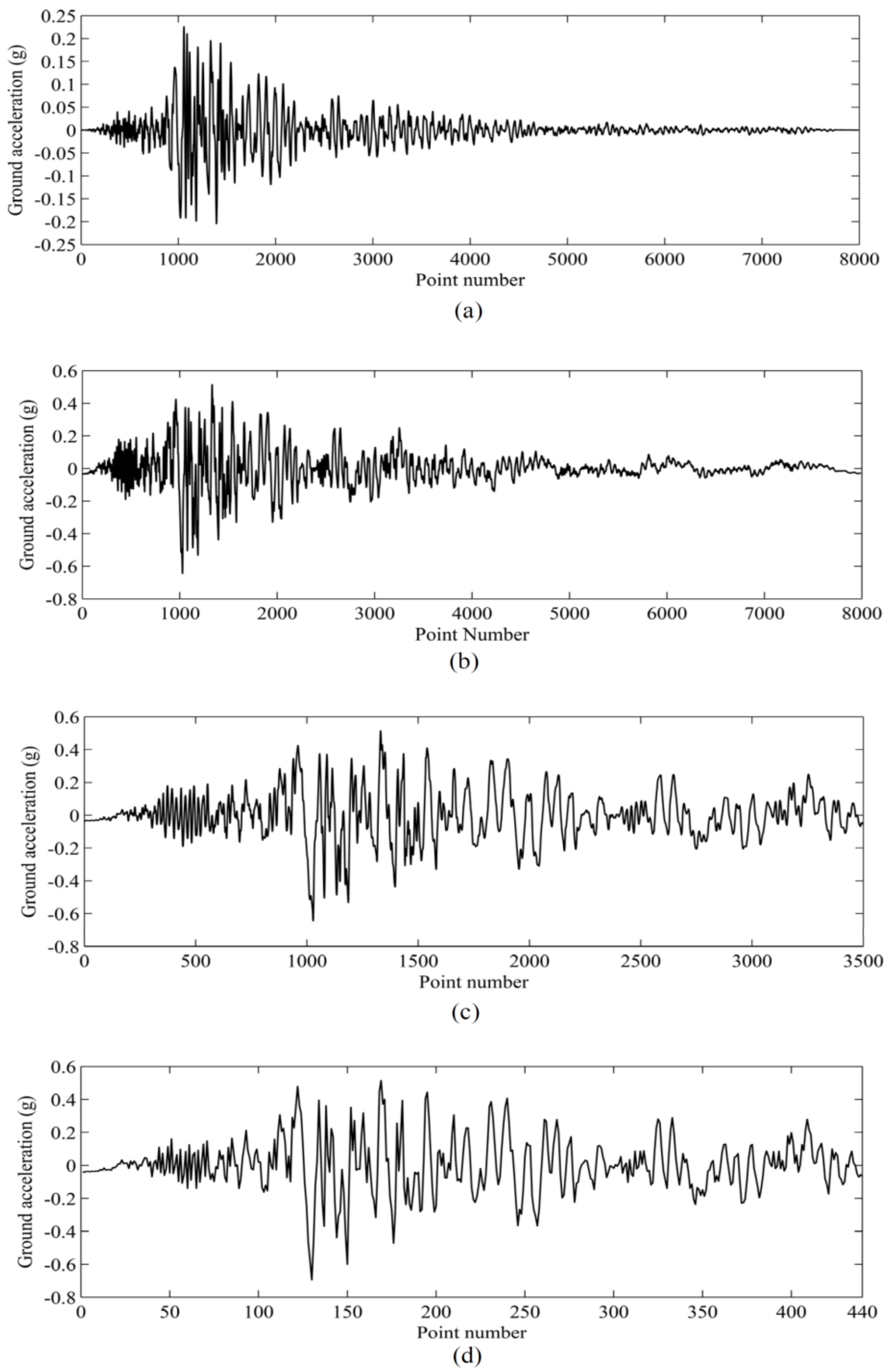

Fig. 6 Loma Prieta ground motion (station: Gilroy Array #7, 1989): (a) Original record; (b) MCE response spectrum matched artificial record; (c) Artificial record in the effective duration; (d) Scaled filtered record for MCE hazard level

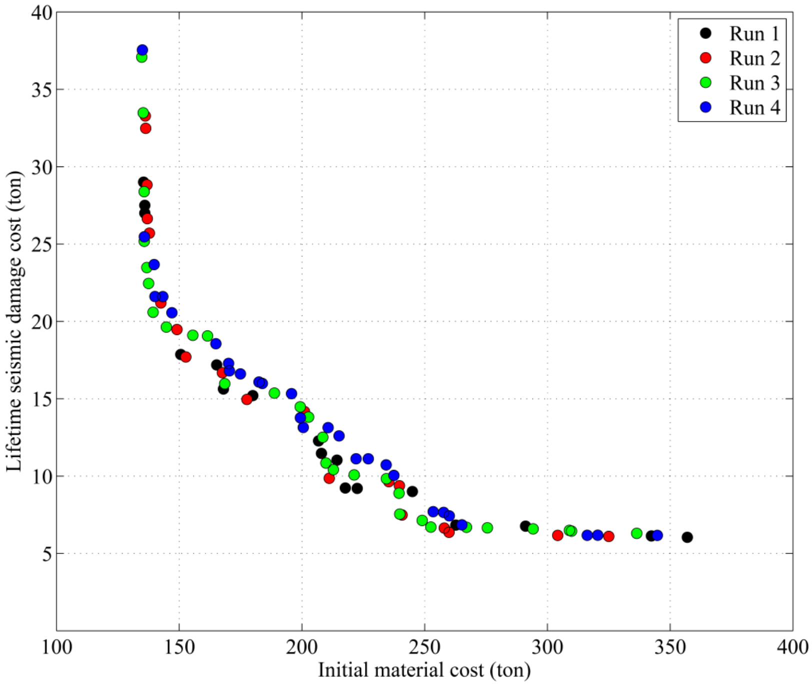

Fig. 7 Obtained Pareto fronts in four different runs for the reported frame

Table 2 Properties of two characteristic designs of the reported frame

	Cross section number												
Group no.	1	2	3	4	5	6	7	8	9	10	11	12	13
Design A*	W530 ×74***	W530 ×66	W840 ×226	W360 ×72	W360 ×72	W530 ×74	W920 ×390	W920 ×390	W920 ×390	W920 ×368	W1000 ×296	W690 ×350	W690 ×265
Design B**	W840 ×392	W920 ×345	W1000 ×296	W360 ×33	W920 ×271	W920 ×970	W920 ×970	W920 ×558	W920 ×558	W1000 ×883	W1000 ×883	W1000 ×883	W1000 ×591

	Fitness function evaluation			
	1st objective function	2nd objective function	Initial cost improvement	Seismic damage cost improvement
Design A	135.02 ton	37.54 ton	62.18%	-521.52%
Design B	356.98 ton	6.04 ton	-164.39%	91.83%

* Indicates the design with minimum initial cost

** Indicates the design with minimum seismic damage cost

*** Units are in SI system

about 696 hours. If none of these simplifying strategies are adopted, the solution algorithm requires 43 times more computational time. These values are estimated by calculating the computational time required for the fitness evaluation of a single design alternative in the cases that one or both of the simplifying strategies are not adopted, and multiplying it by the product of

the population size and the number of generations of the genetic algorithm.
In order to compare the properties of the different optimal designs achieved in the Pareto fronts, two characteristic designs are investigated. These designs are the extreme points correspond to the single-objective optimal designs where minimization of the initial material cost and the seismic damage cost are respectively the objective functions. The properties of these two designs are listed in Table 2. As shown in this table, the initial material cost of design B is 308% higher than design A, while the corresponding seismic damage cost is 72% lower. The values of C_{LSD} factor computed for the design A and B are respectively 0.2341 and 0.0165.

This paper proposed a framework, in accordance with FEMA-350 specifications, for the performance-based multi-objective optimal design of steel moment-frame structures. Minimization of the life-cycle cost were considered by treating the initial cost and the seismic damage cost as two separate objectives of the optimization problem. A computer program was developed based on the proposed framework and operated for the design of a ten-story steel moment-frame structure. Obtaining the Pareto front of the possible optimal designs of a structure provides invaluable economical information that helps investors or insurance companies to make the best decisions. They can select among the Pareto optimal designs the one that is the most economical in terms of financial resources. This issue is more important specifically in large-scale construction projects. In the present study, we have tried to consider most of the relevant constraints included in the guidelines, so that the results may be useful for the engineers in real-life projects.

For improving the time efficiency of the solution algorithm, two different strategies were adopted. Firstly, a simplified modeling method was employed to reduce the level of structural modeling sophistication needed for the seismic analysis of structures. In this method, a multi-bay steel moment-frame is condensed to a single bay moment-frame with properties tuned to represent the original frame. The simplified models required the solution of significantly fewer degrees of freedom than models with member-by-member representation. Secondly, a three-level wavelet decomposition method was used to decrease the number of involved acceleration points in the time-history loading to 0.125 of the given earthquake record. By using the filtered record, the solution algorithm requires about eight times less computational time.

In order to validate the reliability of the simplifying strategies adopted in this paper, two numerical tests were carried out. In the first test, a ten-story moment-frame was modeled with both the simplified and the member-by-member representation for 100 randomly generated design variables and analyzed against the same earthquake record. In the second test, 100 simplified models for the ten-story moment-frame were generated randomly and analyzed subjected to both the Loma Prieta earthquake record and its surrogate record. The low values of errors in predicting the interstory drift ratios, obtained from the both tests, confirm the accuracy of the methodology developed in this paper. However, the quality of the results is definitely dependent on the topology of structures and for structures with irregularities in plan or in elevation, some additional loss of accuracy is expected.

9. Conclusions

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