Optimal Design of RC Frames Using Nonlinear Inelastic Analysis

Bora Gencturk and Kazi Ashfaq Hossain

Abstract Recent earthquakes, especially those in Chile (2010) and Christchurch (2011), have demonstrated the unexpected performance of buildings designed according modern seismic design codes. These incidents strengthen the cause for moving towards performance-based design codes rather than serviceability and strength design. This chapter deals with optimal design of RC frames, a widely used structural type around the world, considering both the initial cost and structural performance as problem objectives. Initial cost comprises the total cost of materials and workmanship for structural components, while structural performance is measured by a two-level approach. First, each design is checked for acceptability according to existing codes, and next performance is quantified in terms of maximum interstory drift obtained from nonlinear inelastic dynamic analysis. This multi-objective, multi-level approach allows one to investigate the implications of the selection of design parameters on the seismic performance while minimizing the initial cost and satisfying the design criteria. The results suggest that structural performance varies significantly within the acceptable limits of design codes and lower initial cost could be achieved for similar structural performance.

Keywords Reinforced concrete · Inelastic dynamic analysis · Structural optimization · Taboo search · Pareto front

1 Introduction

Structures have been traditionally designed to withstand applied loads and deformations, with appropriate factor of safety, which they may experience throughout their

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service life. These designs are commonly based on code requirements, contemporary practices, and subjective judgment and experience of the personnel involved in the design process. After the 1994 Northridge earthquake in the United States, it has been observed that the structures that comply with the code requirements, although achieved life safety objective, sustained significant damage resulting in major economic losses. The Northridge earthquake has been the main stimulus in the United States for moving towards performance-based seismic design (PBSD) codes. These initiatives resulted in a series of notable documents: ATC-40 [1], FEMA-273 [2] and FEMA-356 [3] which were more recently converted to a standard in ASCE-41 [4].

Cost reduction has always been an objective in engineering design. In a typical residential/office building the cost of structural components is relatively low compared to those of mechanical, electrical, plumbing and non-structural features. Nevertheless, reducing the initial structural cost is important both from financial and sustainability standpoints. The latter is associated with the use of non-renewable earth resources, CO_2 emission and other negative impacts on the environment. This chapter is an attempt to tie cost savings with quantifiable structural performance during earthquakes in an optimization framework for reinforced concrete (RC) buildings. Initial cost is evaluated based on the cost of materials and labor for structural components. Structural response is, on the other hand, quantified using nonlinear inelastic dynamic analysis in order to gain insight on expected performance which goes beyond what existing code regulations can provide. As such, the proposed framework falls within PBSD with an extension to include optimization to understand and guide the decision making process.

After a literature review on cost optimization of RC structures, the subsequent sections outline the essential steps of the proposed framework through an example application: definition of the seismic hazard and selection of earthquake ground motions for nonlinear inelastic dynamic analysis, modeling, structural analysis and evaluation of initial cost, structural optimization, and processing of results and decision making.

2 Literature Review

This chapter concerns the optimal design RC buildings considering the initial cost. As a result, in the following sections, after a brief summary of weight optimization studies, which mainly target steel structures, a detailed review of previous studies on initial cost optimization of RC buildings is provided.

2.1 Weight Optimization

Several studies in literature aimed at minimizing the weight of the structure based on the assumption that the cost is directly proportional to the weight ([5–13] among others). Although this is, for the most part, true for steel structures, it is difficult to make such a correlation for RC structures. Therefore, studies on weight-optimal design of concrete structures are limited in comparison. These studies can be primarily divided into component-level and whole-structure optimization.

Weight optimal design of RC beam elements was performed by Chung and Sun [14]. The beam thickness and reinforcement area were considered as design variables with constraints on deflection, stress, and section sizes. Incremental finite element technique was used to unify structural weight optimization with structural analysis, design, and sensitivity analysis. Sequential linear programming (SQP) algorithm was used to incorporate material nonlinearity in the formulation. Karihaloo and Kanagasundaram [15] used linear and nonlinear programming techniques to solve weight minimization problem of statically indeterminate beams with constraints on normal and shear stresses. While, Karihaloo and Kanagasundaram [16] proposed minimum-weight design of elastic plane frames under multiple loads taking into account the effects of buckling and transverse deflections. Under certain assumptions, the optimization problem was reduced to a non-linear programming (NLP) problem, which was solved using several methods: sequential convex programming (SCP), sequential linear programming (SLP), and sequential unconstrained minimization technique (SUMT).

2.2 Cost Optimization

Although material weight contributes to a major part of the total cost of a structure, weight optimization does not take into account other significant cost components such as labor cost and cost of formwork. Materials are the major cost component for steel structures and the initial cost can be represented in terms of material weight. Unlike steel structures, cost optimization is more appropriate for concrete structures due to use of multiple materials. Hence, costs of concrete, reinforcing steel, labor and formwork need to be considered. Numerous studies have been performed on cost optimization of RC beams, columns, slabs and frames. These studies are grouped based on the number of objectives (single vs. multiple) and the optimization approach (mathematical programming-based, gradient-based or heuristic), and reviewed in this section.

2.2.1 Single-Objective and Mathematical Programming-Based Optimization

The objective function for the single-objective cost optimization problems is typically chosen as the initial cost of the structure comprising material and construction costs. Design variables comprise section sizes and reinforcement ratios for all the members. Various structural performance metrics as defined in the building codes are selected as constraints. Earlier attempts in structural optimization of building frames were more oriented towards the use of non-heuristic optimization techniques.

An exhaustive review of literature on mathematical programming-based optimization can be found in [17].

Mathematical programming methods (or direct methods) are mostly linear and nonlinear programming techniques, which have been successfully applied to cost optimal design of RC structures. These methods were found to perform satisfactorily for limited number of design variables and constraints. Several notable studies used mathematical programming for cost optimization of RC structures [18–28].

2.2.2 Single-Objective and Gradient-Based Optimization

Mathematical programming optimization had less success in addressing feasible solutions for realistic optimization problems. On the contrary, gradient-based methods (or indirect) methods are found to be more efficient for large-scale optimization problems by taking into account numerous design variables and constraints. Use of gradient-based methods requires the existence of continuous derivatives of both the objective function and the constraints. For this reason, in most cases, analytical formulations are adopted to evaluate performance metrics. Below is a review of selected studies on optimization of RC structures using gradient-based methods.

Cheng and Truman [29] developed a framework for optimal design of RC and steel structures using optimality criteria (OC) approach. Structural assessment was performed using elastic static and dynamic analysis. In order to meet the requirement of the used optimization algorithm, discrete member properties were converted to continuous variables. Structural weight (or cost) was chosen as the objective function subject to constraints on displacements. Moharrami and Grierson [30] used OC method to determine the optimum cross-sectional dimensions and longitudinal reinforcement of the components of RC buildings subject to constraints on strength and stiffness. Costs of concrete, steel and formwork formed the objective function. Performance of the structure under gravity and static lateral loads was considered and evaluated based on the prevailing code requirements. The results indicated that OC method converges smoothly to least-cost design and the final design is independent of the initial selection of the design variables.

Adamu and Karihaloo [31, 32] used discretized continuum-type optimality criteria (DCOC) for cost minimal design of RC beams with freely varying or uniform cross-sections along the span. Limiting values were applied on deflections, bending and shear strengths with bounds on design variables. The results were compared with those computed using continuum-type optimality criteria (COC) in another paper [33]. In a separate study the authors used the same criteria for RC frames with columns under uniaxial and biaxial bending actions [34, 35]. Design variables included width and depth of the members and reinforcing steel ratio. Deflection, bending and shear strengths were chosen as constraints. Fadaee and Grierson [36] investigated the effects of combined axial load, biaxial moments and biaxial shear on three-dimensional RC elements. OC method was used for optimizing the sections sizes and reinforcement areas. Chan [37] investigated optimal lateral stiffness design of tall RC and steel buildings using the OC method. The objective was to minimize the cost subject to lateral drift, stiffness and serviceability constraints. Constructability and practical sizing of members were also taken into consideration. The proposed method was applied to an 88-story building.

Chan and Zou [38] utilized the principle of virtual work to generate elastic and inelastic drift response of RC building. Response spectrum and nonlinear pushover analyses were used respectively to produce those responses. The formulation was based on OC approach. A two-phase optimization approach was adopted. In the first phase, optimum member sizes were obtained through elastic design optimization. In the second phase, reinforcement ratios were found for previously determined sections through inelastic design optimization. In another study, Zou and Chan [39] used OC method to minimize the construction cost of RC buildings subject to constraints on lateral drifts. Response spectrum and time history loading were applied based on Chinese seismic design code. Lateral drift response was formulated based on the principle of virtual work. Multiple earthquake loading conditions were taken into consideration for optimal sizing of members. Chan and Wang [40] investigated the cost optimization of tall RC buildings subject to constraints on maximum lateral displacement and interstory drift. Member sizes were designed based on OC approach. Zou [41] proposed an optimization technique for base-isolated RC buildings based on OC method. Similar to the author's previous studies, lateral drift response was formulated based on the principle of virtual work. The underlying assumption of this study was that all the members of the superstructure behave linear elastically while the isolation system behaves nonlinearly.

2.2.3 Single-Objective and Heuristic Optimization

In spite of being computationally efficient, gradient-based approaches have limited scope because both the zeroth and at least first order derivatives of the objective function and the constraints are needed. In addition, the search domain needs to be continuous, which prevents the use of discrete design variables such as the reinforcing steel areas (or ratios). To circumvent these problems, researchers used the method of virtual work to explicitly define the objective function and constraints. The review in the previous section indicates that OC was preferred as the gradient-based optimization algorithm in most studies. Recent advancement in computational tools, on the other hand, enables researchers to include computationally costly analysis methods, such as static pushover analysis and dynamic time history analysis in structural optimization problems, through finite element modeling. However, in most cases conventional gradient-based algorithms cannot be used because the continuity of functions or their derivatives may not exist. By using heuristic approaches, this problem can be overcome. Furthermore, heuristic algorithms can effectively find global minima, while gradient-based algorithm might be trapped at a local minimum.

Genetic algorithm (GA) was first used as a technique to solve engineering optimization problem by Goldberg and Samtani [42]. Based on his study, many researchers successfully employed GA for design optimization of structures. A comprehensive review of studies related to structural optimization based on GA is available in [43]. Choi and Kwak [44] created a database of different RC sections sorted from the least to most resistance for obtaining optimum member design. A two-step algorithm, which involved finding the continuous and discontinuous solution from the database, was used. Design variables were reduced to a single one by using section identification numbers. Optimization of the entire structure was proposed by combining individually optimized elements. Similarly, Lee and Ahn [45] developed a data set containing section properties of frame elements in a feasible range while performing discrete optimization of RC plane frames based on GA. The semi-infinite search space was converted to a finite one by using the data sets, which were further modified and reduced based on the provisions of existing code regulations on reinforcement area and configuration. Camp et al. [46] investigated material and construction cost minimization of RC frames based on GA. Serviceability and strength constraints were used to satisfy the code requirements that are incorporated in the algorithm as penalty functions.

Balling and Yao [47] used a multi-level approach for design optimization of RC concrete frames. RC frame optimization was identified to be more complicated than steel frames because of the problems with reinforcement design. The optimization of reinforcement detailing was simultaneously conducted with the optimization of cross-sectional dimensions. This approach enabled the investigation of the effect of reinforcement topology, bar selection, bar positioning, cutoff and bend points, and stirrups and ties. A simplification was made based on the assumptions that either the lower bound of reinforcement area or strength would govern the optimum design. Similarly, Rajeev and Krishnamoorthy [48] considered discrete design variables for detailing and placing of reinforcement in RC frames as opposed to traditional practice of selecting steel area as continuous design variables that required rounding up to realistic constructible values.

Govindaraj and Ramasamy [49] studied the cost optimal design of continuous RC beams based on GA. Only the cross sectional dimensions of beams were considered as design variables in order to reduce computational costs. Constraints were applied on strength, serviceability, ductility, durability as per Indian standards. Detailing of reinforcement was accounted for in a sub-level optimization problem. Saini et al. [50] performed cost-optimal design of singly and doubly reinforced concrete beams subjected to uniformly distributed and concentrated loads based on artificial neural networks (ANN). To bypass trapping of ANN in local minima, GA was used to optimize the architecture and user defined parameters. The limit state design and the optimization were performed with constraints on moment capacity, actual deflection and durability along with other geometric constrains according to Indian standards.

Sahab et al. [51] proposed a two-stage hybrid optimization algorithm based on modified GA and applied this algorithm to perform cost optimization of RC flat slab buildings. In a similar study, Sahab et al. [52] presented multi-level optimization procedure for RC flat slab building. Column layouts along with section sizes and number of reinforcing bars were obtained through exhaustive search, whereas the hybrid optimization algorithm was used to find section sizes. Constraints were applied based on the design regulations. In a different study, in order to reduce the computational costs in finding optimal design of structures subjected to earthquake loads, Salajegheh et al. [53] combined two artificial intelligence strategies: radial basis function (RBF) neural networks and binary particle swarm optimization (BPSO), and proposed a hybrid optimization method.

Leps and Sejnoha [54] implemented augmented simulated annealing method for optimizing shape, bending and shear reinforcement of RC structures, simultaneously. An example was presented for a continuous beam. Rao and Xiong [55] proposed a new hybrid GA where GA was applied to determine the feasible search region that contains the global minimum. The optimum solution was obtained through an integrated algorithm comprising hybrid negative sub-gradient method and discrete one-dimensional search. An example was presented for optimal design of an RC beam. Ahmadi-Nedushan and Varaee [56] applied Particle Swarm Optimization (PSO) method to one-way RC slabs with different support conditions. The total cost of the slab was selected as the objective function subject to constraints on strength, ductility and serviceability as recommended in the design code. A dynamic multi-stage penalty function was chosen which transforms the constrained problem to an unconstrained one by penalizing the impractical points on the search space. El Semelawy et al. [57] found optimum values of slab thicknesses, number and sizes of tendons, and tendon profiles of pre-stressed concrete flat slabs based on modern heuristic optimization techniques. A general and flexible tool was developed that could handle real life problems. Costs of concrete and tendons were included in the objective function. Results suggested that the consideration of a second objective function (distance from constraints) would make the optimization technique more efficient.

Fragiadakis and Papadrakakis [58] studied deterministic and reliability based optimization for designing RC frames against seismic forces and found the latter to be more feasible in terms of economy and flexibility of design. Non-linear response history analysis was performed for structural performance assessment. The objective was to obtain improved performance against earthquake hazards with minimal cost. Evolutionary algorithm (EA) was used to solve the optimization problem. Three hazard levels and several limit states from serviceability to collapse prevention were considered. In order to reduce the computational time, fiber-based beam-column elements were used only at the member ends, and inelastic dynamic analysis was performed only if non-seismic checks performed through a linear elastic analysis were met.

2.2.4 Multi-Objective Optimization

In most studies on single-objective optimization, the merit function was selected to minimize the cost of the structure through optimal material usage. Alternative designs were explored to obtain the optimal solution. Hence, single-objective optimization methods usually provide just one optimal solution. Decision makers either have to accept or reject the optimum design. On the other hand, multiple merit functions, which are related to decision making process, are taken into consideration in multi-objective optimization. It offers decision makers the flexibility to select the "best" (or most suitable) option from a number of equivalent solutions based on their priorities and judgments. Hence, several studies formulated multi-objective optimization problem by modifying existing algorithms to account for multiple objective functions.

Ang and Lee [59] formulated an integrated framework for optimization of RC buildings with respect to minimum life-cycle cost criteria. Life-cycle cost included initial costs from materials, labor, and construction together with probable damage cost from future earthquake hazards. By applying the minimum life-cycle cost criteria, constraints for the allowable risk of fatality were measured. Li and Cheng [60] incorporated damage-reduction based structural optimization algorithm into seismic design of RC frames. Initial costs and total expected loss formed the objective function. A simplified approach for reliability analysis was adopted along with a tailored enumeration technique. Findings included improved seismic performance of damage-reduction-based design over traditional design, on the grounds of several metrics such as life-cycle cost, structural responses against extreme earthquakes and reliability of the weakest story based on the drift.

Lagaros and Papadrakakis [61] compared two design approaches: based on European seismic design code and performance-based design (PBD) for three-dimensional RC frames. The considered two objective functions were the initial construction cost and maximum inter-story drift. Linear and nonlinear static analyses were performed for European code based and PBSD, respectively. Three performance objectives corresponding to three hazard levels were considered. EA was used for optimization. Design based on Eurocode was found to be more vulnerable to future earthquakes. Zou et al. [62] used OC method to minimize the initial material cost and life-cycle damage cost of RC frames in a multi-objective optimization framework for performance based earthquake engineering (PBEE). Optimal member sizes were determined through elastic response spectrum analysis in the first stage of optimization. In the second stage, static pushover analysis was performed to find the reinforcement ratios. Fragiadakis and Lagaros [63] presented an alternative framework for PBSD of structures. Particle swarm optimization algorithm was adopted. The formulation could account for any type of analysis procedure (linear or nonlinear, static or dynamic). Initial cost or lifetime seismic loss could be selected individually or together to define the objectives of the problem. Both deterministic and probabilistic design procedures were incorporated. A number of limit states from serviceability to collapse prevention were selected for probabilistic design.

Paya et al. [64] used cost, constructability, sustainability (environmental impact), and safety as the four objective functions while performing structural optimization of RC frames based on multi-objective simulated annealing (MOSA). Design was performed according to Spanish code. Pareto optimal set of solutions were obtained. Mitropoulou et al. [65] used life-cycle cost assessment (LCCA) to evaluate the designs based on a prescriptive and performance-based methodology. Initial construction cost was minimized in the former case; while, in the latter case, life-cycle cost was considered as an additional objective function, turning the problem into a multi-objective one. Incremental dynamic analysis (IDA) and nonlinear static pushover

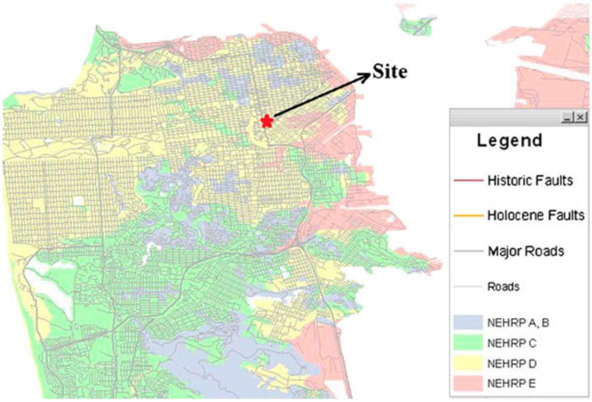

Fig. 1 Soil profile in San Francisco Bay area [68]

analysis were performed for structural assessment. Various sources of uncertainty were taken into consideration for seismic demand and structural capacity.

3 Seismic Hazard and Earthquake Ground Motions

3.1 Definition of Seismic Hazard

For the example application in this study, the design spectrum is derived according to ASCE 7-10 [66]. A site is selected at the intersection of Market Street and Van Nesse Avenue in San Francisco, California with coordinates 37° 46′ 29.67″ N, 122° 25′ 10.12″ W. The soil type at the location is determined as site class D according to the NEHRP [67] scale as shown in Fig. 1. Design spectral response acceleration parameters at short periods, S_{DS}, and at a period of one second, S_{D1}, are taken as $1g$ and $0.6g$, respectively. The design response spectrum is computed based on ASCE 7-10 section 11.4.5. The calculated design spectrum, shown in Fig. 2, is used for selecting the earthquake ground motions as described in the next section.

3.2 Selection and Spectrum Matching of the Earthquake Ground Motions

For nonlinear inelastic analysis of the structural frames, the seismic response history procedures of ASCE 7-10 are followed. Accordingly, three earthquake ground motions are selected from the database of Pacific Earthquake Engineering Research (PEER) Center [69]. In order to conform to the requirements of ASCE 7-10 on

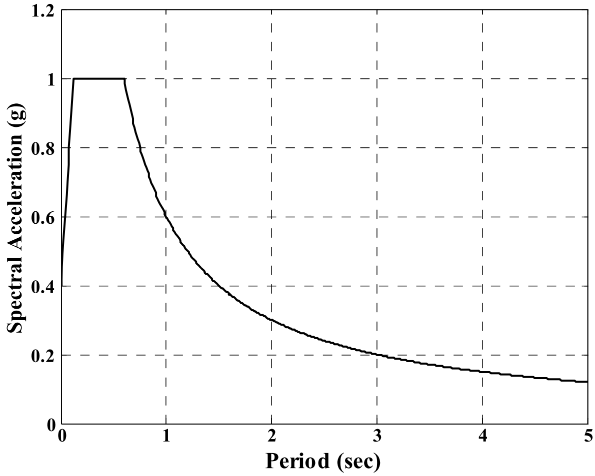

Fig. 2 Design spectrum at the selected site

ground motion selection, spectrum matching is utilized. The fundamental period of the frames change during optimization because of the changing decision variables (i.e. section sizes and reinforcement ratios). Based on eigenvalue analysis of a typical frame considered in this example (see Sect. 4.1), the fundamental period is estimated as 0.5 seconds. ASCE 7-10 requires the ground motions be scaled such that the average value of the five percent damped response spectra for the suite of motions is not less than the design response spectrum for periods ranging from $0.2T$–$1.5T$, where T is the fundamental period of the structure. For the typical frame mentioned above, the period range suggested by ASCE 7-10 is 0.1–0.75 seconds. Given that the fundamental period of the frames will change during the optimization process, spectrum matching is performed for 0–1 seconds to be conservative. Note that spectrum matching (rather than acceleration scaling) is utilized here as this approach results in less record-to-record variability in structural response [70, 71].

In spectrum matching process, target spectrum is defined as the ASCE 7-10 design spectrum for the period range from 0 to 1 seconds while the spectra of the original ground motions are retained for larger periods. This approach ensures that unrealistic high period oscillations are not introduced into the spectrum compatible ground motions by matching outside the period range that is relevant to the frames considered here. Spectrum matching is performed using the modified version of the RSPMatch software [72] described in [70]. The plot of design spectrum along with the spectra of the selected ground motions before and after spectrum matching is shown in Fig. 3, while the acceleration time histories are shown in Fig. 4. The spectrum compatible time histories in Fig. 4(b) are used for nonlinear inelastic dynamic analysis.

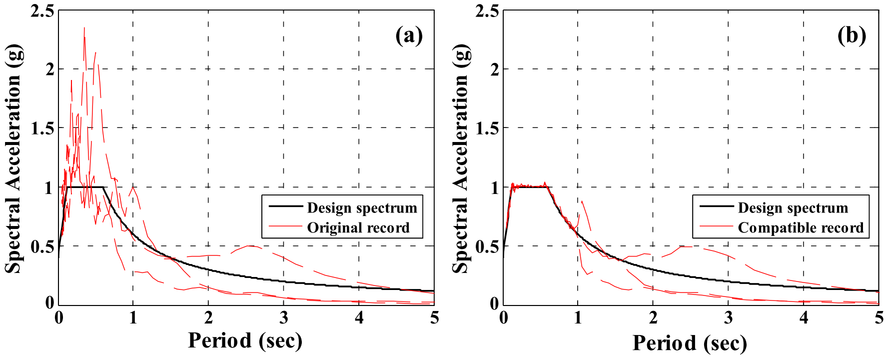

Fig. 3 Acceleration response spectrum of (**a**) original, and (**b**) spectrum compatible ground motion records

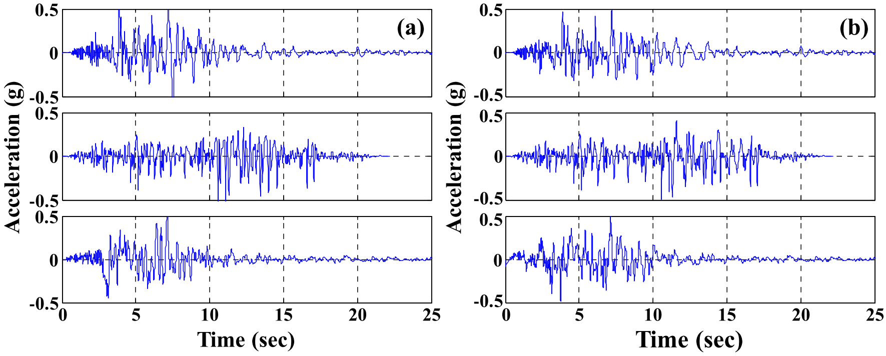

Fig. 4 (**a**) Original, and (**b**) spectrum compatible ground motions

4 Modeling, Structural Analysis and Initial Cost

4.1 Structural Frames

The four-story three-bay reinforced concrete structural frame, shown in Fig. 5, is chosen for performing the design optimization. Story height and bay width of the frame as well as the initial cross sections and initial reinforcement ratios of all the members are selected based on the archetype design (ID 1008) in [73]. Initial column sizes are 558.8 mm × 558.8 mm (22 in × 22 in) with total longitudinal reinforcement ratios varying from 1.13 % to 1.63 % depending on the story level and column type (exterior or interior). Typically, reinforcement ratios are higher in the interior columns. All beams are assigned a cross sectional dimension of 558.8 mm × 609.6 mm (22 in × 24 in). Unlike the original archetype design, same reinforcement ratios are used at both tension and compression sides of a beam which are reduced gradually with increasing floor level from 0.83 % (first floor level) to 0.45 %

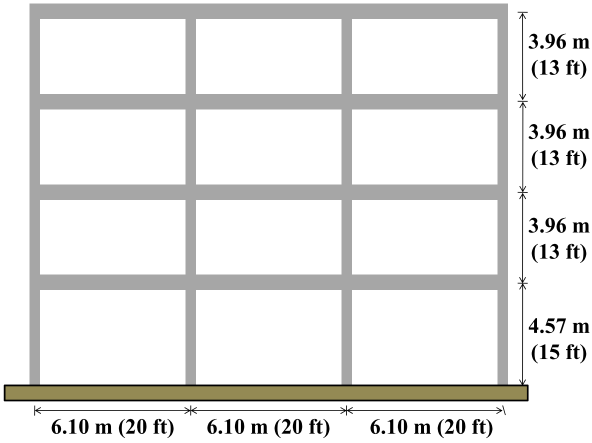

Fig. 5 The structural frame used for the example optimization problem

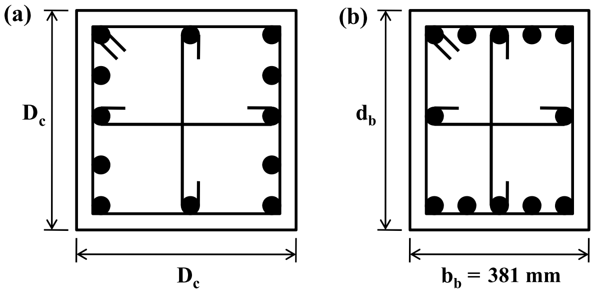

Fig. 6 Typical cross sections of (**a**) columns and (**b**) beams

(roof level). Beam stirrups and column stirrups are spaced at 127 mm (5 in) with total reinforcement ratios of 0.33 % and 0.7 %, respectively. The design inter-story drifts at each story ranged between 0.6 % and 1.2 %.

In order to minimize the search space, only the most important parameters are taken into consideration. The following assumptions are made in selecting design variables are: (i) all columns have the same dimensions, (ii) constant reinforcement ratio is maintained throughout a beam or a column, (iii) beam depths and reinforcement ratios change every two floors, (iv) beam width at every floor is fixed to 381 mm (15 in), and (v) shear reinforcement ratios or stirrups are not considered as design variables; rather these are designed based on elastic analysis. Most of these assumptions are also needed for construction feasibility and applied in real projects. It is also assumed that all beams and columns contain fixed number of longitudinal and transverse bars with predefined reinforcement configurations. The column and beam sections used for optimization are shown in Fig. 6.

Based on these assumptions, seven design variables are selected: column width, reinforcement ratios of exterior and interior columns, depth and reinforcement ratio of first two story beams, and depth and reinforcement ratio of top two story beams. A range of discrete values is assigned for each of these design variables, which are listed in Table 1. The bounds of these values comply with ACI 318-11 [74]. All

Table 1 Design variables and ranges for the considered structural frames

Design variables	Values
Width of columns (mm)	381, 508, 635, 762, 889, 1016
Reinforcement ratio of external columns	0.01, 0.02, 0.03, 0.04, 0.05, 0.06
Reinforcement ratio of internal columns	0.01, 0.02, 0.03, 0.04, 0.05, 0.06
Depth of first two story beams (mm)	381, 508, 635, 762, 889, 1016
Reinforcement ratio of first two story beams	0.005, 0.01, 0.015, 0.0175, 0.02, 0.025
Depth of top two story beams (mm)	381, 508, 635, 762, 889, 1016
Reinforcement ratio of top two story beams	0.005, 0.01, 0.015, 0.0175, 0.02, 0.025

possible combinations of these design variables generate 279,936 cases, which set up the search space for this optimization problem.

The gravitational loads for the considered space (interior) frame include a floor dead (including self-weight) and live loads of 8379 N/m^2 (175 psf) and 2394 N/m^2 (50 psf), respectively. The equivalent lateral load method, which is one of the recommended procedures of ASCE 7-10 [66], is used for defining the earthquake loads in elastic analysis. Lateral loads are computed according to ASCE 7-10 section 12.8.1. The response modification factor (R), overstrength factor (Ω_0), and deflection amplification factor (C_d) corresponding to a special moment resisting frame are used, which are 8, 3 and 5.5, respectively. Effective seismic weight of the frame is taken as the full dead load plus 25 % of the live load. Based on the seismic design coefficients and the seismic weight, design seismic base shear is found to be 360.3 kN (81 kips). The total base shear is then distributed at each floor level by assuming an inverted triangular (code suggested) distribution. The approximate fundamental period of the structure for the initial section sizes and reinforcement ratios is calculated as 0.58 seconds.

4.2 Evaluation of Structural Capacity and Earthquake Demand

Structural capacity and earthquake demand are evaluated using two different analysis techniques. A linear elastic analysis is performed and design checks are made according to ACI 318-08 [75] and IBC 2009 [76]. All the load combinations (including the seismic effects) stipulated in these regularity documents are taken into account. Additionally, P-Delta effects are accounted for in the analysis and design checks. The structural capacity is not measured based on a specific response quantity; on the contrary, a combination of decision variables is categorized into a binary variable of acceptable/unacceptable (pass/fail) based on the serviceability and

Table 2 Cost items

Item	Unit	Cost ($/unit)
	Material costs	
Steel (longitudinal), A615 grade 40	metric ton	1018.5
Steel (transverse), A615 grade 40	metric ton	1253.8
Concrete, ready mix (35 MPa)	m^3	145.2
Cast-in-place concrete forming	m^2	29.6
	Labor costs	
Placing steel (longitudinal) in beams	metric ton	806.9
Placing steel (transverse) in beams	metric ton	2050.3
Placing steel (longitudinal) in columns	metric ton	948.0
Placing steel (transverse) in columns	metric ton	2182.6
Placing concrete	m^3	64.2
Placing concrete forming	m^2	110.3

strength checks. If a combination of decision variables does not satisfy any of the code requirements it is classified as unacceptable.

Earthquake demand on the other is evaluated through a nonlinear inelastic dynamic time history analysis using the fiber-based finite element analysis program ZEUS NL [77]. The structural frames are modeled using displacement-based beam-column elements with cubic shape functions [78, 79]. Concrete [80] and reinforcing steel [81] are modeled using the existing models ZEUS NL materials library. Geometric nonlinearity is taken into account in the dynamic analysis. A response history analysis is performed under each of the three spectrum compatible earthquake records shown in Fig. 4(b) and the earthquake demand is measured in terms of the maximum absolute interstory drift at any of the columns. Interstory drift is selected here as the response metric to measure the earthquake demand for being closely related to the development of P-Delta instability (a system level indicator), and to the amount of local deformation imposed on the vertical elements and beam column connections (component level indicators).

4.3 Calculation of Initial Cost

The cost of materials (concrete, reinforcing steel and formwork) and labor (placing) are considered here for the initial cost calculation of the structural frames. The cost items are estimated based on 2011 Building Construction Cost Data [82] and provided in Table 2. The details of initial cost calculation can be found in [83].

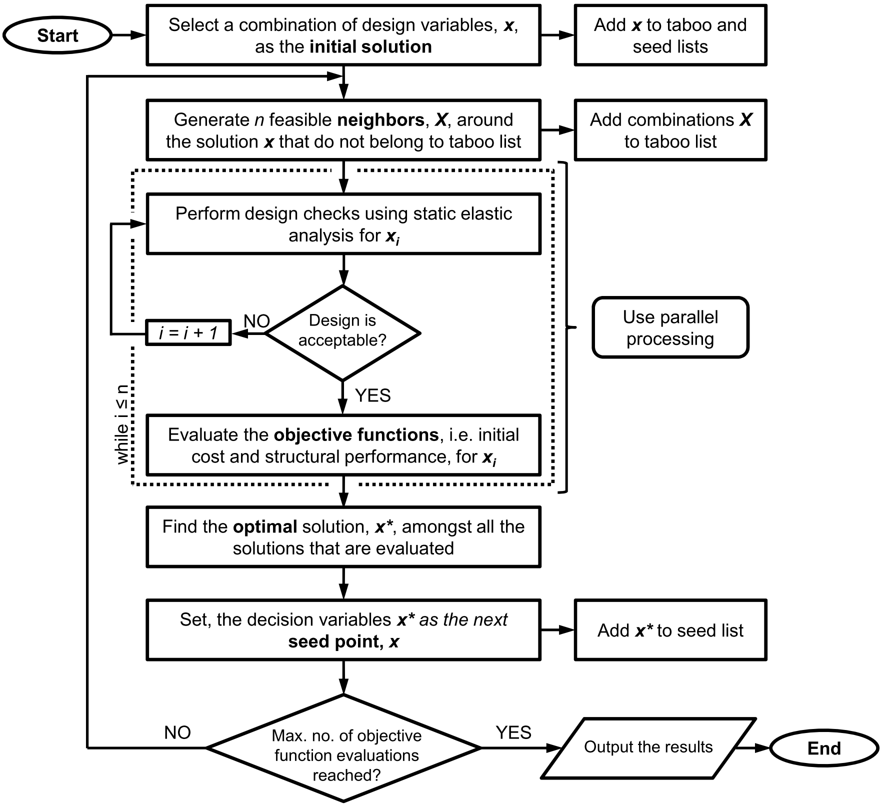

Fig. 7 Flowchart of the structural optimization approach

5 Structural Optimization

As discussed earlier, design optimization of RC structures is a challenging task, especially when inelastic dynamic analysis is used to evaluate the performance metrics. To partially overcome this problem a two level approach is adopted here. First, an elastic static analysis is performed for a combination of design variables. All the design checks, as mentioned in Sect. 4.2, are performed according to selected regulatory documents. If the design is classified as acceptable, inelastic dynamic time history analyses are performed under the three spectrum compatible ground motions. The maximum of the maximum interstory drifts obtained from the three analyses is used as one of the objectives (structural performance). On the contrary if the design is classified as unacceptable, the structural performance objective is set to a large value so that this combination of design variables is penalized and not further considered by the optimization algorithm. The other objective is defined as the initial cost. Both objectives are only evaluated if the selection of design variables is acceptable. The flowchart of the optimization procedure is provided in Fig. 7.

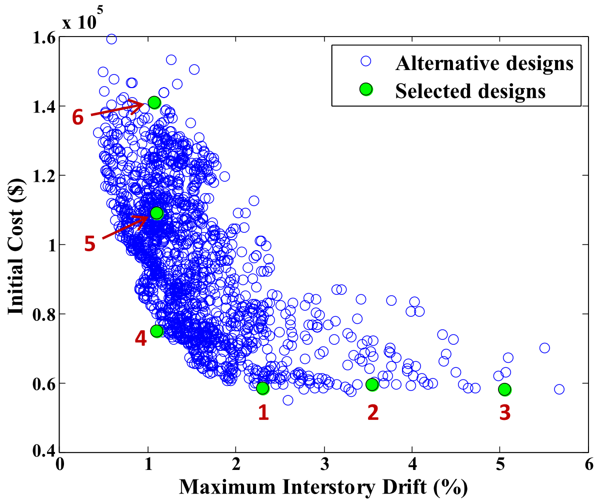

Fig. 8 Results of multi-objective optimization in the solution space

Taboo search (TS) algorithm is used to obtain the optimal solutions for the multi-objective optimization problem considered here. TS algorithm has been applied to various structural optimization problems and it has been showed to be very effective in solving combinatorial optimization problems with nonlinear objective functions and discontinuous derivatives [83–86]. TS employs a neighborhood search technique to sequentially move from a combination of design variables $\boldsymbol{x}$ (i.e. section sizes and reinforcement ratios) that has a unique solution $\boldsymbol{y}$ (initial cost and maximum interstory drift), to another in the neighborhood of $\boldsymbol{y}$ until some termination criterion is reached. To explore the search space, at each iteration TS selects a set of neighboring combinations of decision variables using some optimal solution as a seed point. Usually, a portion of the neighboring points is selected randomly to prevent the algorithm being trapped at a local minimum. TS algorithm uses a number of memory structures to keep track of the previous evaluation of objective functions and constraints. The most important memory structure is called the taboo list, which temporarily or permanently stores the combinations that are visited in the past. TS excludes these solutions from the set of neighboring points that are determined at each iteration. The existence of the taboo list is crucial to the optimization problem considered here because the evaluation of objective functions and/or constraints are computationally costly.

6 Results and Discussion

The results of structural analysis are shown in the solution space (initial cost vs. maximum interstory drift) in Fig. 8. Note that each circle in Fig. 8 represent a combination of the decision variables and all the points satisfy the design checks according to ACI 318-08 [75]. Before all, the most important conclusion from these

Table 3 Cost and performance comparisons of alternative designs

Design numbering according to Fig. 8	Design variables							Total cost ($)	Maximum interstory drift (%)	Fundamental period (sec)
	Depth and width of columns (mm)	Reinforcement ratio of external columns	Reinforcement ratio of internal columns	Depth of first two story beams (mm)	Reinforcement ratio of first two story beams	Depth of top two story beams (mm)	Reinforcement ratio of top two story beams			
1	508	0.02	0.02	508	0.025	508	0.01	58544	2.308	1.01
2	508	0.03	0.03	508	0.0175	381	0.015	59595	3.543	1.06
3	508	0.02	0.02	635	0.015	381	0.01	57922	5.054	0.94
4	635	0.02	0.02	762	0.0175	635	0.015	74754	1.097	0.62
5	889	0.06	0.01	635	0.0175	635	0.0175	109111	1.102	0.54
6	1016	0.05	0.04	635	0.01	1016	0.005	140966	1.077	0.42

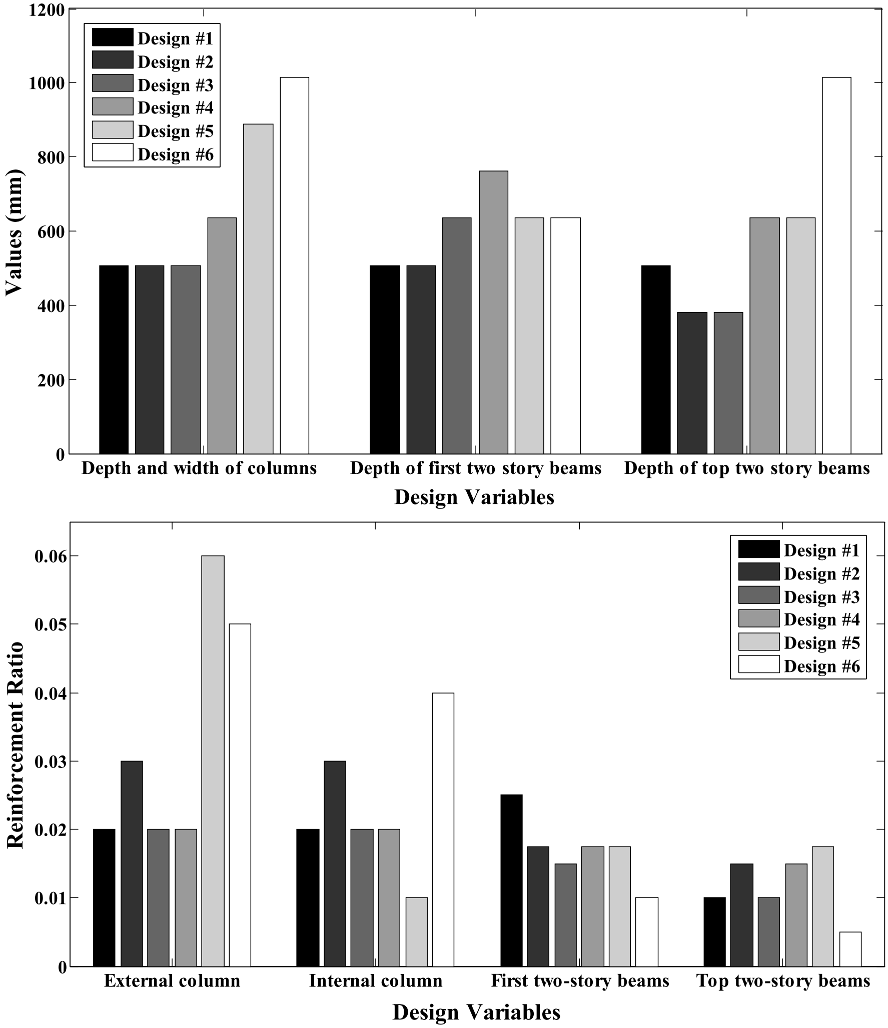

Fig. 9 Comparison of design variables for the similar cost and similar performance cases

results is that although the design satisfies the code regulations, significant variation in structural performance could be observed depending on the selection of design variables. In several cases the maximum interstory drift reaches or exceeds five percent, which can be detrimental for the vertical load carrying system and may result in partial or total collapse of the building if sufficient ductility and load carrying capacity at large displacement are not provided. Notwithstanding significant changes to seismic design codes since 1994 Northridge earthquake, these results suggest that even if we can ensure collapse prevention in most cases, there is a good chance that we will observe unexpected structural performance by a number of structures which

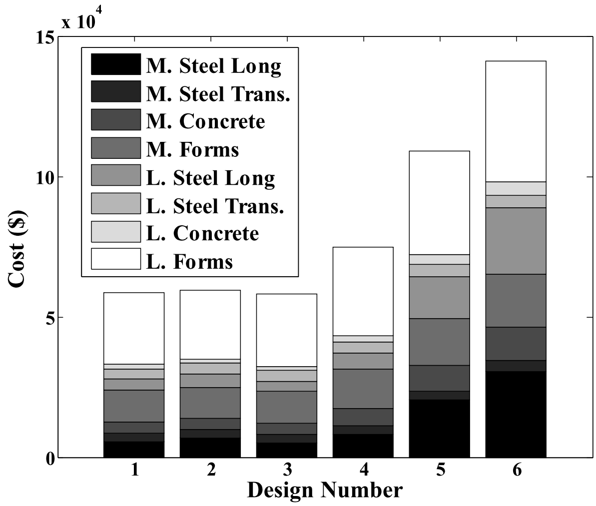

Fig. 10 Breakdown of initial cost: designs 1–3 and 4–6 show similar cost and similar performance cases, respectively

may have severe consequences such as deaths, injuries, and direct and indirect economic losses.

Among all the alternative designs, six are selected in order to assess the contribution of each design variable to the total cost or structural performance. The selected solutions are identified in Fig. 8 and corresponding decision variables are shown in Table 3 along with total cost and maximum interstory drift. For the first three designs (rows of Table 3), the total initial costs are very close, whereas maximum interstory drifts vary significantly. While for the last three designs, the opposite is true. The comparison of design variables for the selected solutions is provided in Fig. 9. For the first two designs, the dimensions of all the members are identical except for the depth of the top two story beams, which is observed to affect significantly the maximum interstory drift. Similarly, for third design, the critical parameter that governs the performance is the beam sectional property (i.e. reinforcement ratio) in top two stories. Apparently, reinforcement ratios of the columns do not have much impact on the drift performance in these cases. It can be observed from the cost breakdown shown in Fig. 10 that each cost component contributes consistently to the total cost across these design alternatives.

The last three design alternatives have very similar drift performances while the total costs are significantly different. These results again confirm that a decrease in section dimensions or reinforcement ratios of columns can be compensated by increasing the depth of the beams (see Fig. 9). These results also shed light on selection of proper design variables in seismic design for both performance and cost. It is clear from Fig. 10 that there are significant differences in total cost, mostly due to material and labor cost of longitudinal steel, although all three design alternatives results in similar structural performance. The labor cost of formwork contributes the most to the total initial cost in all the cases; on the contrary, the labor cost of concrete has the lowest contribution. Finally, it is seen in last column of Table 3 that the fundamental period of the frames changes considerably by changing decision variables; however, it still remains within 0–1 seconds, which is the range selected for spectrum matching of earthquake ground motions.

7 Future Research Directions

Optimal seismic design of RC building is a challenging task, mainly due to associated computational cost and difficulty in defining performance objectives. In addition to section sizes, and reinforcement ratios, reinforcement topology needs to be optimized as well. With increasing design variables, the search space grows rapidly resulting in excessive computational demand, especially when rigorous structural analysis approaches such as the nonlinear inelastic dynamic analysis is used. Thus, there is a need to develop methods for identifying design variables that are practical for engineers and that govern the seismic response.

Other performance definitions such as lifetime economic losses, deaths and injuries should also be included among the objectives of the optimization problem. It is likely that these additional layers of computation will reduce the efficiency of optimization algorithms; therefore, proper techniques should be developed to test and overcome these potential problems. The uncertainty in structural response due to sources such as the variation in material properties, effects of non-structural components, errors in analysis models and inherent randomness in ground motion processes should also be incorporated in structural optimization.

8 Conclusions

In this chapter we have attempted to lay out a framework for cost optimal seismic design of RC buildings. The novelty of the proposed approach is the multi-objective, multi-level investigation. The former is by consideration of both the initial cost and structural performance, while the latter is due to a two-level structural assessment where each design alternatives is first assessed for code compliance and then performance is quantified using rigorous structural analysis. The framework is applied to a four-story three-bay RC building. The results support the potential use of the proposed approach in decision making process where both initial cost and structural performance is considered at the same time while complying with the code requirements. The shortcomings of the approach mainly stemming from the high computational demand and negligence of uncertainty need to be addressed in future studies.

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