

## OPTIMAL PERFORMANCE-BASED SEISMIC DESIGN OF COMPOSITE BUILDING FRAMES WITH RC COLUMNS AND STEEL BEAMS

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### ABSTRACT

Composite RCS building frames integrate reinforced concrete columns with structural steel beams to provide an efficient solution for the design and construction of earthquake-resisting structures. In this paper, an optimization framework is developed for performance-based seismic design of planar RCS moment resisting frames. The objective functions are defined as minimizing the construction cost and the seismic damage. The design variables are obtained in a two-stage design optimization procedure; the elastic design in which column cross-section dimensions are determined and the inelastic design in which beam cross-sections and column reinforcements are obtained. Two design examples are presented to demonstrate the applicability and efficiency of the proposed method. Based on the obtained results, it is concluded that the proposed design optimization procedure is a viable approach in producing cost effective seismic designs of composite RCS frames, with reliable seismic performance and reduced damage potential in the event of a severe earthquake ground motion.

**Keywords:** optimization; performance-based design; seismic design; composite structure; RCS frame, colliding bodies optimization.

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### 1. INTRODUCTION

Steel-concrete composite structures are becoming increasingly popular as an alternative to bare steel structures in moderate to high seismic zones, due to their efficiency and construction economy. Composite moment resisting frames with reinforced concrete (RC) columns and steel beams, so called RCS frame systems, have been widely used in the United States [1] and Japan [2]. The systems in the US typically consist of a perimeter (two-

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dimensional) frame, which is made up of RC columns (with small embedded steel erection columns for construction purposes) and structural steel beams running through the columns. With the aid of structural steel skeleton, speed of construction is achieved by spreading the construction activities vertically, such that different trades are engaged simultaneously in the construction of the building. The RCS frames in Japan, have been applied as an alternative to conventional reinforced concrete construction, but with some differences in construction methods and the structural forms, compared to the US practice (steel erection columns are not used in Japanese practice, and the beams are connected to the columns in two directions, generating a space frame). In general, the RCS composite frame is a more economical structural system compared to traditional all-steel or all-reinforced concrete frame, since the most efficient use of steel, reinforced concrete, and composite members is realized in a composite structural system.

Extensive research has been conducted on the behavior and design of RCS frame structures. An overview of cooperative research program among U.S. and Japan can be found in [3]. Wei et al. provide a state-of-art report on the seismic performance of RCS frames and their subassemblies [4]. Full-scale experimental investigations reported by Cordova and Deierlein [5] indicate that the RCS frame satisfies seismic performance criteria under various seismic hazard levels. Steele and Bracci [6] evaluated the performance and constructability of RCS systems for use in low- to mid-rise frame buildings. Mehany and Dierlin [7] investigated the seismic performance of RCS frames using nonlinear static and time-history analyses. They concluded that RCS frames exhibited excellent seismic performance, especially those with through-beam type connections where the fracture critical details of welded steel moment frame are avoided by permitting steel beams to run continuous through the reinforced concrete columns.

Based on the results of the above-mentioned research stream, it is indicated that composite RCS frames designed in accordance with the modern code provisions can provide cost-effective solutions with reliable seismic performance under multi-level earthquake hazards. Using the insight provided by the aforementioned researches, it is promising to develop a general design philosophy that enables designers and owners to select higher performance levels in order to limit property and business interruption losses while looking for more cost effective designs. Such design methodology is termed performance-based seismic design (PBSD) that is rapidly gaining acceptance in professional practice. Using PBSD, structures can be designed to particular damage levels for different earthquake ground motions. PBSD formulated in the context of a structural optimization problem is a topic of growing interest and significant research studies have been conducted in recent years, regarding for example steel structures [7-12], reinforced concrete structures [13-16] and bridge piers [17-20].

A few publications have appeared in the literature dealing with design optimization of composite buildings [21-24]. These research studies, however, do not follow the performance-based design philosophy. Recently, the author presented a structural optimization framework for performance-based seismic design of composite frames with concrete-filled steel columns and steel beams [25]. Using the proposed optimization procedure, composite frame designs with minimum construction cost and minimum seismic damage may be generated. In this paper, the proposed optimization framework is extended to performance-based seismic design of RCS frames. The main distinction is the selection

and determination of the design variables. In an RCS composite frame, the design variables include member sizes and reinforcement ratios of the reinforced concrete columns and section sizes of the steel beams. Following the procedure developed by Zou and Chan for RC frames [16], the design variables are determined in a two-stage optimization algorithm. In the first step, an elastic design optimization is performed to find member sizes of concrete columns and steel beams. In fact, minimum required stiffness of the frame is obtained under serviceability seismic actions assuming an elastic response.

Once the optimal structural member sizes are determined at the end of the first step of the optimization, the second step, i.e. the inelastic design optimization is carried out to determine the column reinforcement ratios as design variables. The steel beam section sizes are also taken as design variables and are updated during the optimization process. The column cross-section dimensions, however are fixed during the second step of the optimization. Two design examples are presented to illustrate the feasibility and efficiency of the proposed optimal design procedure.

## 2. PERFORMANCE-BASED SEISMIC DESIGN OF BUILDING STRUCTURES

Performance-based seismic design is the modern conceptual approach to structural design, which is based on the principle that a structure should meet performance objectives for multiple seismic hazard levels, ranging from small magnitude earthquakes of a short return period, to more intensive events with long return periods. A performance objective is a combination of performance levels each linked to a specific hazard level.

### 2.1 Seismic hazard levels

Seismic hazard is generally specified as the probability of exceedance of a certain hazard level or alternatively, the average return period for a given value of seismic hazard (e.g., ground acceleration or spectral acceleration). ATC-40 [26] specifies three levels of earthquake ground motion having 50%, 10% and 5% probability of exceedance in 50 years (mean return periods of approximately 75, 500 and 1000 years, respectively). These specified earthquake intensities are termed Serviceability earthquake, Design earthquake and Maximum earthquake respectively. FEMA-356 [27] defines Maximum Considered Earthquake (MCE) hazard as a 2% probability of exceedance in 50 years (return period of approximately 2500 years). Based on these specifications, three levels of earthquake hazard are considered in this study for the design of composite moment frames, namely SE, DE and MCE with 50%, 10% and 2% probability of exceedance in 50 years (mean return periods of approximately 75, 500 and 2500 years), respectively. The elastic spectral response acceleration for each hazard level may be obtained from the 5% damped spectral function as

$$S_a^i = \begin{cases} F_a^i S_s^i \left[ 0.4 + \frac{3T}{T_s^i} \right] & 0 \leq T \leq T_0^i \\ F_a^i S_s^i & T_0^i \leq T \leq T_s^i \\ \frac{F_b^i S_1^i}{T} & T_s^i \leq T \end{cases} \quad i = SE, DE, MCE \quad (1)$$

where  $T$  is the elastic fundamental period of the structure computed from structural analysis.  $S_s^i$  and  $S_1^i$  are mapped short-period response acceleration parameter and mapped response acceleration parameter at one-second period, respectively for each hazard level  $i$ .  $F_a^i$  and  $F_v^i$  are site coefficients and  $T_0^i = 0.2T_s^i$  and  $T_s^i = S_1^i/S_s^i$ .

## 2.2 Performance levels

The performance level can be defined as specified limits on any response parameter such as stresses, strains, displacements, accelerations, damage states or the failure probability [28, 29]. Various definitions and specifications of performance levels are introduced in the literature. According to FEMA-350 [30], four building performance levels are defined including Operational (OP), Immediate Occupancy (IO), Life safety (LS) and Collapse Prevention (CP) performance levels. These performance levels may be correlated with minor or no damage, moderate damage, severe damage and near collapse damage states, respectively. Structural damage is often quantified using global response parameters such as roof drift ratio (RDR) and inter-story drift ratio (ISDR). Allowable RDR limits for IO, LS and CP performance levels are suggested as 0.7%, 2.5% and 5%, respectively [31]. ISDRs corresponding to IO and CP levels are given in FEMA-350 (for low-rise buildings: 1.25% and 6.1% respectively). Based on the results of analyses and experiments on the performance of composite structures [32], acceptable LS performance under the DE earthquake is obtained with maximum RDR and ISDR limits of 2% and 3%, respectively. Satisfactory CP performance under MCE earthquake is realized when the roof and inter-story drift ratios are limited to 4% and 5%, respectively. Minor or no damage state for OP performance is achieved if the RDR and ISDR is limited to 0.4% and 0.5% respectively. These maximum allowable values of drift ratio are summarized in Table 1.

Table 1: Maximum allowable values of response parameters

Response parameters	Performance level		
	Operational (OP)	Life Safety (LS)	Collapse Prevention (CP)
RDR	0.4%	2%	4%
ISDR	0.5%	3%	5%

## 2.3 Performance objective

In this research, an enhanced performance objective is considered for the design of RCS composite moment resisting frames. This enhanced objective is met by achieving the OP performance level under the SE hazard level, LS performance level under the DE hazard level, and the CP performance level under the MCE hazard. Table 2 indicates the performance objective intended for the design.

Table 2: Performance objective

		Performance level		
		Operational (OP)	Life Safety (LS)	Collapse Prevention (CP)
Earthquake hazard level	SE (50%/50)	✓		
	DE (10%/50)		✓	
	MCE (2%/50)			✓

### 3. MODELING AND ANALYSIS PROCEDURE

#### 3.1 Modeling of structural components

Fiber model developed by Taucer et al. [33] for reinforced concrete beam-column elements is used for modeling of RC columns. The model is based on a distributed plasticity formulation where the nonlinear inelastic response of the member is distributed along its length and cross-section. The rectangular reinforced concrete cross-section is discretized into fibers of the corresponding concrete and steel materials as shown in Fig. 1. Behavior of each fiber is governed by the uniaxial stress-strain relationship of the particular material. Monotonic stress-strain ( $\sigma$ - $\epsilon$ ) curves for unconfined concrete, confined concrete and steel fibers are plotted in Fig. 2.

Beam members are simulated using simple lumped plastic hinges assigned to the ends of the elements. Nonlinear characteristics of the deformation controlled hinges are obtained from the material and geometric properties of the corresponding element cross-section.

Details of the composite beam-column connections are shown in Fig. 3. These connections are the so-called "through-beam" type where the steel beams are continuous through the column and spliced away from the column face. By avoiding interruption (splicing) of the beam at the point of maximum moment, the fracture-critical joints typical of conventional steel construction are eliminated. As discussed by Cordova et al. [34], the joints within the RCS frames are inherently strong and are not expected to experience any significant deformations and thus their contribution to the overall system behavior is considered as negligible. Therefore, in this study the joints are modeled as rigid connections.

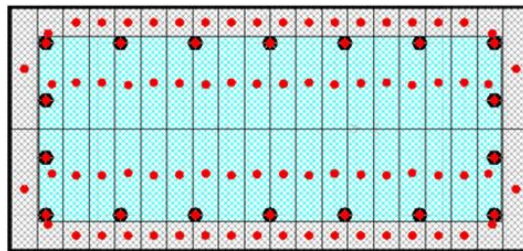


Figure 1. Fiber element discretization of reinforced concrete cross-section

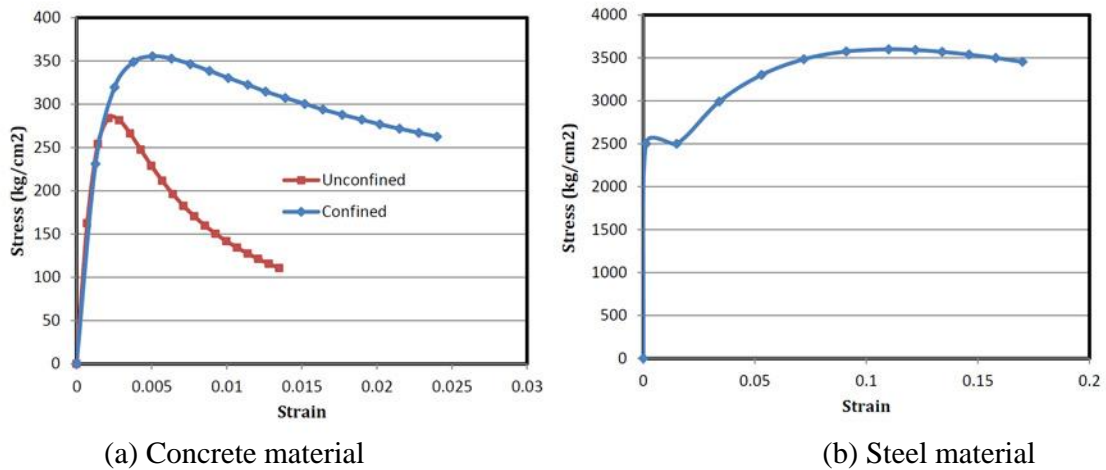


Figure 2. Material properties

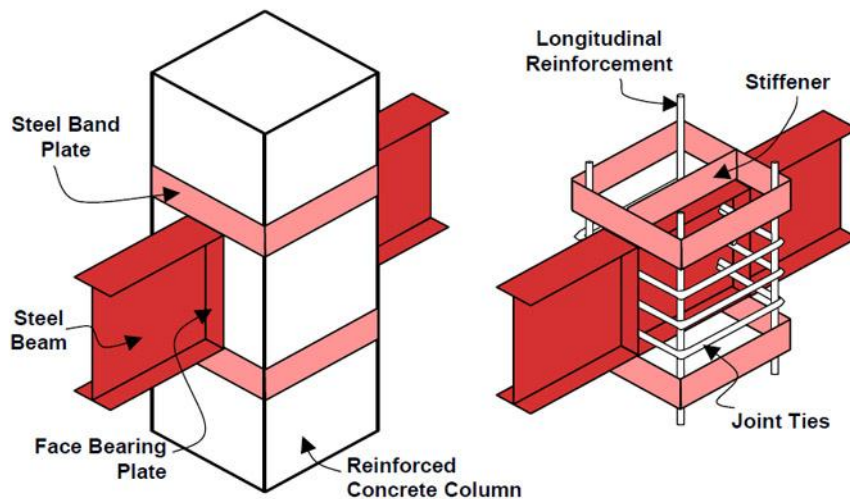


Figure 3. Through-beam type RCS frame connection [5]

### 3.2 Structural analysis procedures

In the first stage of the design optimization of RCS frames, the structure is assumed to behave linear elastic under serviceability seismic actions and consequently a linear response spectrum analysis is adopted in the first step. Concrete columns are assumed to be uncracked and their stiffness is determined only by their section dimensions. Thus, column reinforcement ratios are not required in this stage.

For the second stage of the algorithm, a load-control pushover analysis (so called spectrum-based analysis in the literature [35]) is adopted. In this method, the analysis is terminated when the maximum specified design base shear is achieved. The design base shear for a specified earthquake hazard level is determined using a site specific design spectrum. The method is an adaptive analysis in that the applied load pattern and the load increments continually change depending on the instantaneous dynamic characteristics of

the system. Similar to the linear response-spectrum analysis method the load pattern in this pushover method can consider as many modes as deemed important during the course of the analysis. Hence, the effect of higher modes can be incorporated in the analysis and design.

#### 4. FORMULATION OF STRUCTURAL DESIGN OPTIMIZATION PROBLEM

##### 4.1 Design variables

Design variables include section dimensions and reinforcement ratio for RC-columns and section sizes for beam elements. Rectangular cross sections with a fixed width (350 mm) and variable length (ranging from 350 mm to 1200 mm with 10 mm increments) are considered for column members. This configuration is selected because the frames being investigated in this study are two-dimensional and the columns are subject to single bending about the major axis of their cross sections. Column reinforcement ratio is considered as a continuous design variable ranging from 0.01 to 0.04. The computed reinforcement area is assumed to be distributed uniformly around the cross section. Beam members are selected from a discrete database of standard rolled steel cross-sections ranging from IPE-120 to IPE-600.

The design variables are determined in two design optimization phases. In the first phase, column cross-section dimensions are determined in an elastic design process subject to serviceability drift constraints, then they are fixed during the next phase. Beam cross-section sizes are also determined in the same process, however they are used as the initial values for the next phase and are allowed to be altered subsequently. The second phase of the algorithm is an inelastic design process, in which column reinforcements and beam section sizes are determined subject to life safety and collapse prevention drift constraints and uniform ductility constraint. Serviceability drift constraints are re-applied in the second phase in order to prevent the violation of operational performance requirements due to beam cross-section updates.

##### 4.2 Objective functions

Minimizing the Construction Cost is the primary objective of many structural optimization algorithms. However, in the context of performance-based design, minimum seismic damage under earthquake loading is an equally favorable objective. In this study, two objective functions concerning construction cost and seismic damage are incorporated into design.

Construction cost is considered proportional to the materials cost of the structural members. In an RCS composite structure, the materials include in-situ concrete, reinforcing steel for rebars and structural steel for rolled shapes. Total material costs of a composite RCS structure may be denoted by the general expression

$$C_{total} = C_{ss} \cdot M_{ss} + C_{sr} \cdot M_{sr} + C_c \cdot V_c \quad (2)$$

where  $C_{ss}$  and  $C_{sr}$  (\$ per ton) are the average total unit costs for structural steel and steel rebars, respectively and ( $C_c$  \$ per m<sup>3</sup>) is the average unit cost of in-situ concrete.  $M_s$  and  $V_c$  are the total steel mass and concrete volume, respectively. By introducing two cost ratio

coefficients as  $CR_r$  (unit cost of steel rebars over unit cost of structural steel) and  $CR_c$  (unit cost of concrete over unit cost of structural steel), the total cost can be replaced by the equivalent steel mass  $M_{s(total)} = M_{ss} + CR_r \cdot M_{sr} + CR_c \cdot V_c$ . To facilitate the optimization process,  $M_{s(total)}$  is divided by the maximum available total equivalent steel mass  $M_{s(max)}$ , and a normalized form of the cost function  $f_1$  is obtained as

$$f_1 = \frac{1}{M_{s(max)}} (M_{ss} + CR_r \cdot M_{sr} + CR_c \cdot V_c) \quad (3)$$

where,  $M_{s(max)}$  is obtained by selecting the upper-bound section sizes and reinforcement ratios for all structural members.

The implementation of optimization algorithm involves two design stages in which the individual terms of the general formulation of cost function presented in equation 3 should be calculated specifically, according to the design variables involved. In the first stage, only concrete column and steel beam section sizes are governed and we are not concerned with determining the column reinforcements. However, in order to prevent the optimization algorithm from choosing the cheaper large size cross-sections for concrete columns, the rebar mass  $M_{sr}$  is calculated using the lower bound 1% reinforcement ratio for all columns. In the second stage, the minimum cost is determined by the amount of steel materials and the RC-column section dimensions are not updated during the optimization process. Therefore,  $V_c$  is not varied between the trial solutions and appears as a constant in the general formulation 3.

Another important objective in performance-based design concerns minimizing earthquake damage. Damage is quantified by relating it to inter-story drift distribution at extreme performance levels, such as CP level. Since uniform ductility demand over all stories generally avoids local weak-story collapse, minimum damage objective is interpreted as providing a uniform inter-story drift distribution over the height of the building. To facilitate the structural optimization process, the uniform ductility objective function  $f_2$  is formulated as a uniform story drift distribution normalized by the number of stories,  $ns$ :

$$f_2 = \left( \frac{1}{ns} \sum_{s=1}^{ns-1} \left( \frac{\theta_s}{\theta} - 1 \right)^2 \right)^{1/2} \quad (4)$$

where  $\theta_s$  and  $\theta$  are the drift ratios at story  $s$  and roof level, respectively. In fact, equation 4 formulates the coefficient of variation of story drift ratio along the structure height. Minimizing  $f_2$  under the MCE earthquake ensures a uniform distribution of damage and hence a minimum seismic damage at CP performance level.

### 4.3 Design constraints

#### 4.3.1 Drift constraints

The overall building drift (roof drift) and inter-story drift are constrained under different earthquake hazard levels in order to ascertain the desired performance levels. As noted in section 2.1, roof drift ratio is limited to 0.4%, 2% and 4% under the SE, DE and MCE earthquake hazard levels, respectively. The maximum allowable inter-story drift ratio is set



to 0.5%, 3% and 5% under the SE, DE and MCE earthquakes, respectively. These limits are quantitative measures to ensure OP, LS and CP performance under the respective SE, DE and MCE seismic hazard levels. The uniform inter-story drift distribution at CP performance level is defined as objective function  $f_2$ . This objective ensures a uniform ductility demand along the structure height and thus implies a minimum damage under the severe earthquake hazard level of MCE. In the course of implementation of multi-objective optimization, the uniform ductility objective is incorporated implicitly by considering it as a constraint bound by some allowable level. This technique is further elaborated in section 4.4.

#### 4.3.2 Strong column/weak beam (SC/WB) constraint

SC/WB concept is advocated in seismic provisions as a means to achieve higher levels of safety and energy dissipation by avoiding soft story mechanism. In this design philosophy, columns are designed strong enough such that flexural yielding generally takes place in beams, leaving the columns virtually free from the formation of plastic hinges except at the base of ground floor columns. Following the proposed Blue Book provision for the SC/WB criterion by SEAOC Seismology Committee [36], we implemented the following constraint applied to all floors except at the roof level:

$$\frac{\sum M_c}{\sum M_b} > 1.0 \quad (5)$$

where,  $\sum M_c$  is the sum of moment capacities of all the columns below the specified level and  $\sum M_b$  is the sum of moment capacities at each end of all the beams of the given floor. As indicated by Cordova [5] the provision 5 is more appropriate to prevent a story mechanism, since it ensures that the columns over the entire floor can provide enough strength to cause hinging in the steel beams.

#### 4.4 Optimization algorithm

The design optimization formulation includes two distinct objective functions namely the minimum structural weight (cost) and the minimum structural damage (uniform ductility distribution). Such a design problem is called multi-objective optimization. Evolutionary algorithms are most suited for solving multi-objective problems due to their population-based search method which allows to find an entire set of Pareto optimal solutions in a single run of the algorithm. Evolutionary optimization algorithms are now well established and successfully applied to different structural optimization problems, as discussed by Kaveh [37, 38]. A number of such algorithms which are developed and elaborated in recent years include Particle swarm optimization (PSO) [39], Ant colony optimization (ACO) [40], Big bang-big crunch (BB-BC) [41], Charged system search (CSS) [42], Ray optimization (RO) [43], Dolphin echolocation (DE) [44], Colliding Bodies Optimization (CBO) [45]. Each of these methods has its own advantages and shortcomings when applied to a particular type of optimization problem. A modified version of CBO algorithm denoted by MCBO is utilized in this paper to solve the optimization problem for performance-based seismic design of RCS composite frames. This method was recently applied to the optimization of post-tensioned concrete bridge superstructures [46], tunnel support linings [47] and performance-based seismic design of quasi-isolated bridge systems [20] and has shown to be

of superior performance and easy to implement. The details of the algorithm and its computer implementation are elaborated in these references and are not restated here for the sake of brevity.

A number of techniques have been developed to deal with the multi-criteria optimization efficiently. Among the different approaches the so-called  $\varepsilon$ -constraint method is employed here due to its consistency with the objective functions employed for the current problem and the simplicity of the method for implementation. The method is based on minimization of the most preferred objective function (here the cost function), while the other objectives (here the uniform ductility objective) are considered as constraints bound by some allowable levels  $\varepsilon$ . Since the optimal value of the objective function  $f_2$  is known (0 for the extreme case of a perfectly uniform inter-story drift distribution), this objective is implemented as a constraint with small bound value  $\varepsilon$  (0.05 for example). In fact,  $f_2$  is thought as the coefficient of variation of the lateral translation distribution as mentioned in section 4.2.

#### 4.5 Overall design procedure

The overall design optimization procedure for an RCS composite frame is listed as follows:

##### First phase- Elastic design optimization

1. INPUT elastic design spectrum for the Service level Earthquake (SE).
2. INITIALIZE design variables including column and beam section sizes ( $x_0$ ).
3. STRUCTURAL ANALYSIS- Perform linear response spectrum analysis using the SE spectrum.
4. OPTIMIZATION
  - Input all necessary design variables and structural response parameters.
  - Check all constraints including drifts and section sizes.
  - Compute the value of cost function.
  - Search for a new improved design  $x_{i+1}$ .
5. CONVERGECE CHECK- Repeat steps 3 to 4 until the change in the value of cost function between two successive design cycles does not exceed a specified tolerance.
6. OUTPUT design information including column and beam section sizes.

##### Second phase- Inelastic design optimization

7. INPUT elastic design spectra for Service level, Design level and MCE earthquakes. Input column and beam section sizes from the previous design phase.
8. INITIALIZE design variables including beam section sizes and column reinforcements ( $x_0$ ). Column section sizes are fixed during the optimization process.
9. STRUCTURAL ANALYSIS- Perform pushover analysis and determine OP, LS and CP performance points.
10. OPTIMIZATION
  - Input all necessary design variables and structural response parameters.
  - Check all constraints including section sizes, roof and story drifts, uniform ductility and SC/WB.
  - Compute the value of cost function.
  - Search for a new improved design  $x_{i+1}$ .

11. CONVERGECE CHECK- Repeat steps 9 to 10 until the change in the value of cost function between two successive design cycles does not exceed a specified tolerance.
12. OUTPUT design information including column and beam section sizes and column reinforcements.

## 5. NUMERICAL EXAMPLES

### 5.1 Example-1: Three-story frame

Consider the three-story, three-span moment frame in Fig. 4. The frame has rigid moment connections, with all column bases fixed at ground level. Based on the tributary areas, the seismic weights are taken as 120 ton for each story. The design variables consist of 3 types of column sections (C1 to C3) and 3 types of beam sections (B1 to B3). Each column type is identified with two design variables: the length of its cross-section in mm and the reinforcement ratio in percentage. The width of the cross-section is assumed to be constant (350 mm) for all column types. The column sections are designated as CXXXXrX.XX, where the first four-digits number indicates the cross-section length in mm, and the second number represents the reinforcement ratio in percentage with two decimal places. Beams are selected from the set of standard rolled steel cross-sections ranging from IPE-120 to IPE-600. The value of concrete cost ratio is estimated as  $CR_c = 0.04 \text{ ton/m}^3$ , which indicates a ‘cheap’ concrete and ‘expensive’ steel (The cost of one ton of steel is approximately 25 times the cost of one cubic meter of concrete). The cost ratio for reinforcement steel is assumed as unity indicating an equal cost of reinforcement and structural steel works. The corresponding upper bound sections (C1200r4.00 for all columns and IPE-600 for all beams) are used to calculate the total maximum equivalent steel mass as  $M_{s(max)}=10.778 \text{ ton}$ , which in turn is used to normalize the cost function  $f_1$ . The uniform ductility objective function  $f_2$  is implemented using the  $\epsilon$ -constraint technique with small bound value  $\epsilon=0.05$ . Yield strength of structural steel and reinforcing steel are assumed as  $f_{ys}=2400 \text{ kg/cm}^2$  and  $f_{yr}=4200 \text{ kg/cm}^2$ , respectively. Concrete compressive strength is assumed as  $f_c=250 \text{ kg/cm}^2$ . Site parameters for constructing the elastic response acceleration spectra at different seismic hazard levels are given in Table 3.

Table 3: Site parameters for design examples

Performance level	Earthquake hazard level	$S_s(g)$	$S_1(g)$	$F_a$	$F_v$
OP	SE	0.27	0.2	1.58	1.99
LS	DE	0.45	0.31	1.44	1.78
CP	MCE	0.7	0.45	1.24	1.55

The results of the performance-based seismic design optimization procedure are summarized in Table 4. The optimal value of the cost objective function is obtained as 0.2565 (i.e., the optimal equivalent steel mass of the frame is  $0.2565 \times 10.778 = 2.765 \text{ ton}$ ).

The normalized base shear – roof drift ratio relationship (pushover curve) for the final design of the frame is plotted in Fig. 5. Three performance levels OP, LS and CP under the corresponding SE, DE and MCE earthquakes are also indicated in the plot. It is observed that

for each performance level, the associated base shear is achieved at the maximum allowable roof drift ratio for that level, i.e. 0.4%, 2% and 4% for OP, LS and CP performance levels, respectively. This implies that the final design obtained by the optimization algorithm provides acceptable ductility capacity under the imposed earthquake demands.

Deformed shapes of the final optimum design of the frame at the corresponding OP, LS and CP performance levels are shown in Fig 6, demonstrating the plastic hinge formation at beams and columns. It is observed from this figure that plastic hinges are confined to the beams and individual columns do not yield except at the base of the ground level columns. It can be argued that the application of SC/WB constraints results in a design which completely eliminates any weak or soft story collapse mechanism. Also shown in this figure are the height-wise distribution of inter-story drift ratio. Fig. 6(c) indicates a uniform distribution of inter-story drift at the CP performance level, as expected.

Table 4: Design optimization results for Example-1

Design variable	C1	C2	C3	B1	B2	B3
Optimization results	C0450r2.39	C0400r2.69	C0350r3.08	IPE-300	IPE-270	IPE-220
Equivalent total structural mass ratio ( $f_1$ )				0.2565		

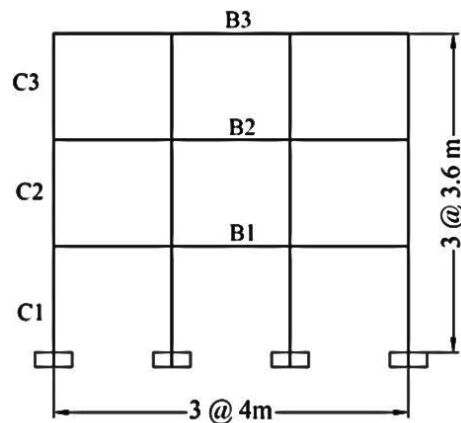


Figure 4. Three-story frame of Example-1

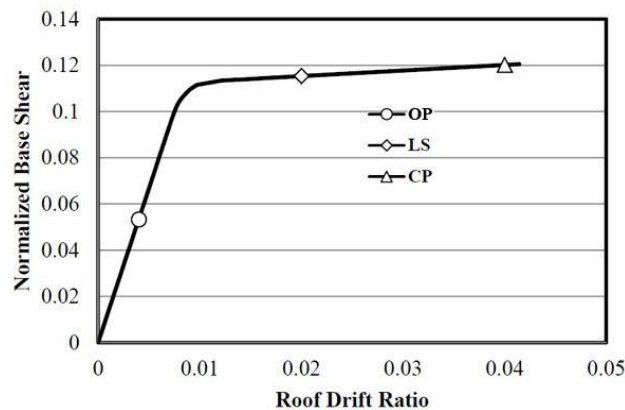
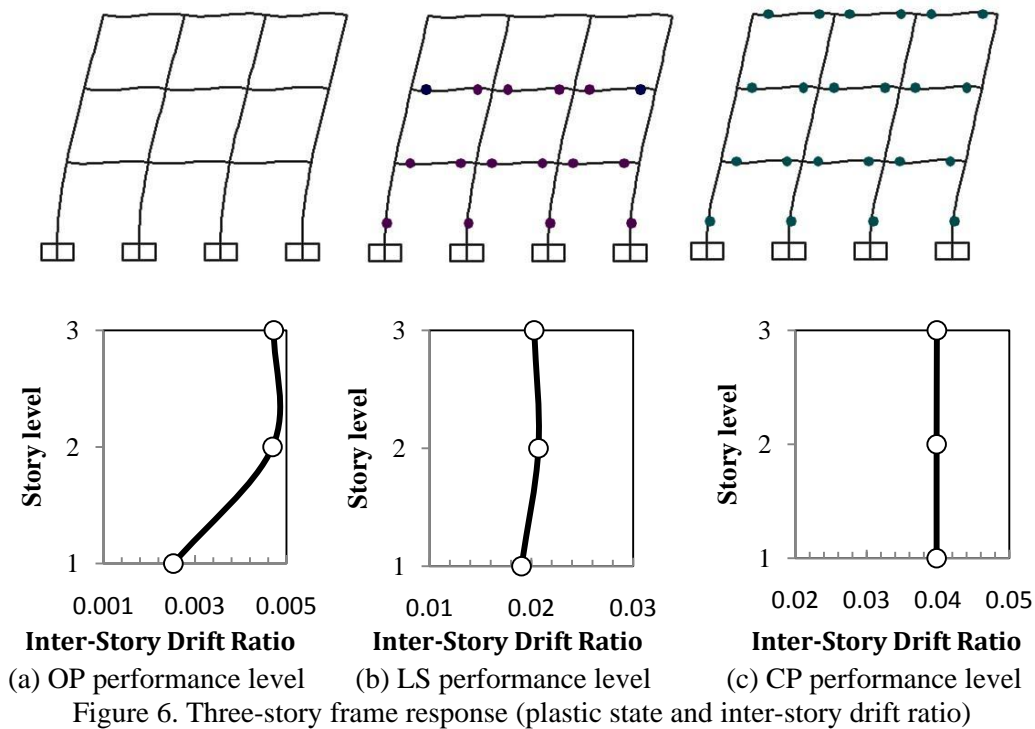


Figure 5. Pushover curve for Example-1



### 5.2 Example 2-nine-story frame

Consider the nine-story, three-span moment frame in Fig. 7. Seismic weight of each story is calculated as 180 ton based on the tributary area. The design variables are reduced to 18, by grouping the columns and beams of each story  $i$  into section types  $C_i$  and  $B_i$ , respectively. The material strength, material cost ratio, design spectral parameters and allowable drift limits are taken to be the same as those of example 1. Total maximum equivalent steel mass of the frame used to normalize the cost function is calculated as  $M_{s(max)} = 38.924$  ton.

The results of the performance-based design optimization procedure are summarized in Table 5. The optimal value of the cost objective function is obtained as 0.4146 (i.e., the optimal equivalent steel mass of the frame is  $0.4146 \times 38.924 = 16.138$  ton)

Fig. 8 presents the normalized pushover curve obtained for the final design of the frame. The three performance points are also indicated in the plot. It is observed that the base shear demands at various performance levels are achieved at the maximum allowed roof drifts, indicating that the optimum design provides adequate ductility capacity.

Fig. 9 present plastic states of the frame at the corresponding OP, LS and CP performance levels as well as the height-wise distribution of inter-story drift ratios. As noted in Example 1, the application of SC/WB constraints results in a frame design in which plasticity is confined to beam members and column yielding takes place only at the ground level. Such a yielding mechanism is more favorable since the risk of a weak or soft story collapse is eliminated. Referring to Fig. 9 it is noted that a perfectly uniform distribution of inter-story drift at the CP performance level is obtained, indicating a uniform damage distribution among the stories.

Table 5: Design optimization results for Example-2

Columns									
Design variable	C1	C2	C3	C4	C5	C6	C7	C8	C9
Optimization results	C0750 r2.39	C0700 r2.31	C0650 r2.48	C0650 r2.21	C0600 r2.39	C0550 r2.61	C0550 r2.28	C0500 r2.51	C0450 r2.79
Beams									
Design variable	B1	B2	B3	B4	B5	B6	B7	B8	B9
Optimization results	IPE-400	IPE-450	IPE-450	IPE-450	IPE-400	IPE-360	IPE-330	IPE-300	IPE-270
Equivalent total structural mass ratio ( $f_1$ )						0.4146			

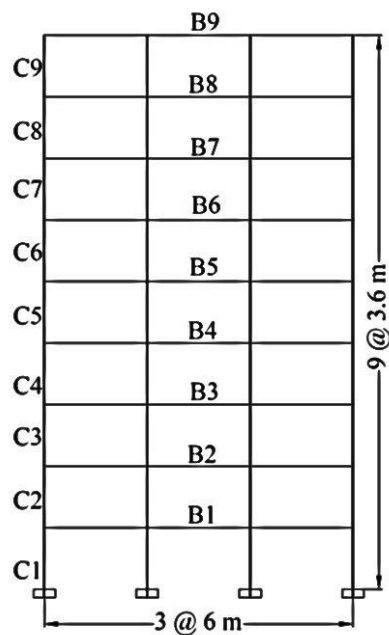


Figure 7. Nine-story frame of Example-2

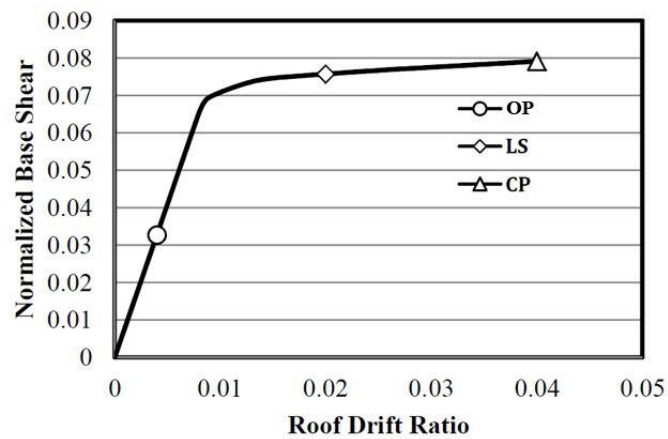
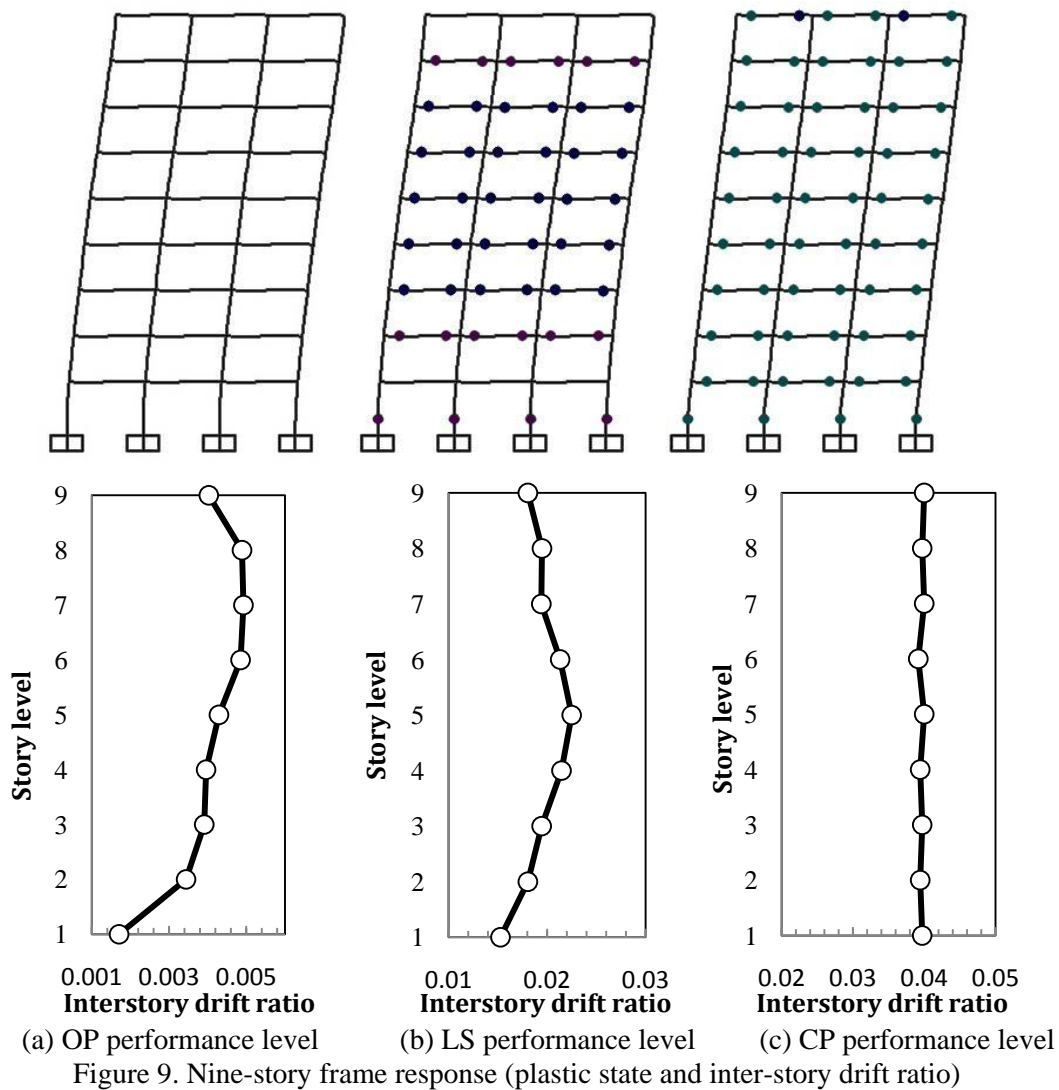


Figure 8. Pushover curve for Example-2



## 6. CONCLUSION

A computer-aided design optimization procedure was developed for performance based seismic design of composite RCS moment resisting frames. Three performance levels of Operational, Life safety and Collapse Prevention under the corresponding earthquake intensities of SE, DE and MCE were considered. Cost and damage were taken as the two objectives to be minimized by the optimization algorithm. The multi-objective optimization problem was formulated and solved by integrating a two-stage analysis-design method, a discrete evolutionary search algorithm (MCBO algorithm) and the  $\epsilon$ -constraint technique.

Two low- and mid-rise moment frames were presented as design examples. The obtained results indicate the feasibility and efficiency of the introduced design optimization approach. Seismic designs of RCS moment frames generated by the proposed method are cost

effective, provide reliable seismic performance and suffer less damage in the case of a severe earthquake ground motion. Although the application of SC/WB constraints together with the uniform ductility objective result in relatively heavier structures, however the optimum designs obtained subsequently have a more reliable seismic performance due to the elimination of any weak or soft story collapse mechanism.

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