PERFORMANCE-BASED OPTIMAL SEISMIC DESIGN OF REINFORCED CONCRETE FRAME BUILDINGS

C. Zhang¹ and Y. Tian²

ABSTRACT

This paper presents an optimal performance-based seismic design approach for multistory reinforced concrete moment frames. The proposed approach minimizes construction cost and takes both inter-story drift and member plastic rotation as optimization constraints. Other seismic design requirements reflecting successful design practice are also incorporated. The optimization contains two stages, the determination of feasible region boundary in strength and stiffness domain and optimization in material consumption domain. The proposed optimization approach is applied to the design of a six-story reinforced concrete frame. The design results indicate that 30% of design earthquake load and 14% of member size can be reduced from the design obtained by the current strength-based approach.

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Performance-Based Optimal Seismic Design of Reinforced Concrete Frame Buildings

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This paper presents an optimal performance-based seismic design approach for reinforced concrete moment frames. The proposed approach minimizes construction cost and takes both inter-story drift and member plastic rotation as optimization constraints. Other seismic design requirements reflecting successful design practice are also incorporated. The optimization contains two stages, the determination of feasible region boundary in strength and stiffness domain and optimization in material consumption domain. The proposed optimization approach is applied to the design of a six-story reinforced concrete frame. The design results indicate that 30\% of design earthquake load and 14\% of member size can be reduced from the design obtained by the current strength-based approach.

\textbf{Introduction}

As a new generation of structural design methodology, performance-based seismic design (PBSD) has been a major focus of earthquake engineering community. PBSD requires that a structure meet target structural performance under various seismic hazards during its design lifetime. Structural performance is generally measured by local and/or global deformations associated with the extent of damage to structural and nonstructural components as well as life and economical losses. PBSD permits designing strength, stiffness, and ductility of a structural system and its components in a coordinated manner, which is difficult to achieve through the conventional strength-based designs. Performance-based seismic evaluation approach for existing builds in the U.S. has been documented in ASCE 41-13 [1] and widely employed to evaluate the seismic performance of existing buildings. For new buildings, however, the conventional strength-based design approach is still prevailing. A major challenge hindering the adoption of PBSD to new building designs is how to handle the vast number of design variables affecting the seismic performance of a structure. Even though PBSD has been occasionally used for new building designs, the typical practice starts from the conventional strength-based design, considering only one level of seismic hazard, and then check the structural performance under

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other levels of earthquake hazard. If needed, the strength-based design results are modified by trial-and-error. This involves significant efforts and the high design cost often deters the building owners from choosing PBSD, even if it can lead to a safe yet economical structural system. Consequently, PBSD has been applied mainly to the design of critical facilities. Because structural performance can be measured by local or global deformations, it is natural to initiate PBSD for a structure using displacement. Due to the requirement of considering multiple levels of seismic hazard and the numerous design variables, there has been an increasing need of optimal PBSD. This paper presents a performance-based optimal seismic design method for RC frames, which synchronizes a new framework for obtaining the optimal design and the most updated formulation of plastic deformation capacity recommended by ASCE 41-13 [1]. The proposed approach is applied to a six-story RC frame.

Statement of Optimal Seismic Design and Optimization Strategy

According to capacity design philosophy, the flexural behavior of RC frame components shall control and flexural strength is the design basis for force-controlled actions such as shear. Thus, this study focuses on the optimal flexural design while minimizing the overall cost associated with concrete and flexural reinforcement consumed by the beams and columns. Multiple levels of seismic hazard are considered, including occasional, rare, and very rare earthquakes. Both inter-story drift ratio \( \gamma \) and plastic hinge rotation angle \( \theta \) are employed to characterize structural response. The overall performance-based optimal seismic design problem can be stated as

\[
\text{Minimize} \quad C_T = c_c \Omega_c + c_s W_s \\
\text{Subject to} \quad \forall \gamma \leq [\gamma], \forall \theta \leq [\theta]
\]

where \( C_T \) is total cost; \( \Omega_c \) is total volume of concrete; \( W_s \) is total weight of flexural reinforcement; \( c_c \) and \( c_s \) are unit costs for concrete and steel reinforcement, accounting for both material and labor; \([\gamma]\) and \([\theta]\) are allowable values for \( \gamma \) and \( \theta \) defined based on target performance. The inter-story drift limit \([\gamma]\) is used to restrain damage to nonstructural components and avoid excessive P-delta effects. \([\gamma]\) is defined as 1%, 2% and 4% for target performance levels of Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP), respectively [2]. Per ASCE 41-13 [1], \([\theta]\) for beams is defined as a function of longitudinal reinforcement ratio and shear. Because the columns are flexure-dominated, their plastic rotation capacity \([\theta]\) is assumed to independent to shear and defined only as a function of axial load.

Fig. 1 outlines the suggested optimization procedure. Eq. (1) incorporates the three major characteristics of an inelastic because \( \Omega_c \) controls elastic stiffness, \( W_s \) is associated with member flexural strength, and \( \gamma \) and \( \theta \) are related to ductility. The two decision variables \( \Omega_c \) and \( W_s \) are constrained by acceptable structural performance. Because the objective function is linear with respect to \( \Omega_c \) and \( W_s \), the optimal solution must be located on feasible region boundary, as shown in Fig. 1(a). Despite the simple format of objective function, obtaining the optimal solution is challenging. First, the peak deformation demand caused by different levels of seismic risks needs to be determined. To permit systematically formulating an optimization procedure, nonlinear static analysis, rather than dynamic analysis, is adopted to predict the seismic response of a RC
The inelastic load-displacement response of the multi-degree-of-freedom (MDOF) system obtained from pushover analysis is transformed into that of an equivalent single-degree-of-freedom (SDOF) system where the spectral displacement is predicted by means of inelastic spectra [3]. The predicted displacement demand is then transformed back to the MDOF system to estimate its peak seismic response. Alternatively, the target roof displacement in the pushover analysis can be determined based on the simplified formulation given in ASCE 41-13 [1].

Figure 1. Framework of optimization: (a) optimization in material consumption domain, (b) stiffness optimization for system with different strengths, (c) MDOF-SDOF transformation, (d) nonlinear static analysis and determination of roof displacement demands, and (e) determination of peak displacement using inelastic spectra

The second challenge is that the peak deformation demand is affected by not only $\Omega_c$ but also $W_s$, implying that the commonly assumed equal displacement rule may not be applicable. As indicated by Krawinkler and Seneviratna [4], for an elastic-perfectly plastic SDOF system, the peak lateral displacement due to ground excitation differs from that of a purely elastic system. The difference is a function of fundamental period $T$, strength reduction factor $R$ (different from that specified in design codes), and ductility ratio $\mu$. These properties are correlated through a $R-\mu-T$ relationship as proposed by Miranda and Bertero [5] and Vidic et al. [6]. It follows that the optimal member size, which controls system elastic stiffness, cannot be determined only from a target lateral displacement without considering the flexural design of members. Accordingly, different from some past studies, $\Omega_c$ and $W_s$ are not individually optimized in this study.

The third challenge of obtaining the solution of Eq. (1) is that the feasible region boundary due to these constraints shall be defined ultimately in the $\Omega_c-W_s$ domain. Nonlinear analysis can correlate structural performance with material consumptions. However, the deformation demands in terms of $\gamma$ and $\theta$ are difficult to be explicitly formulated as a function of $\Omega_c$ and $W_s$. Moreover, $[\theta]$ is affected by shear or axial force, which varies during lateral loading,
suggesting that \( \theta \) of a member is coupled with seismic demand and cannot be predefined. Therefore, a two-stage optimization is considered in this study. In the first stage, a RC frame is designed conventionally based on strength as an initial result. Optimization as shown in Fig. 1(b) is then performed to determine the feasible region boundary defined by a stiffness parameter \( \lambda \) and a flexural strength parameter \( \alpha \) normalized by the initial design. In the second stage, the feasible region boundary determined previously is converted in the \( \Omega_c-W_s \) domain and the optimal solution of Eq. (1) is obtained.

**Additional Design Requirements and Simplifications**

In addition to restraining local and global deformations, other design requirements to ensure desired seismic performance of RC frames are incorporated and enforced from the initial design. To reduce the risk of forming soft stories, the needed flexural strength of a column is determined by the maximum bending moment demand this column may experience when the RC frame is loaded in the nonlinear static analysis up to a target roof displacement due to rare or very rare earthquakes. To further reduce the likelihood of column yielding at a beam-column joint, the needed flexural strength of columns follows ACI 318-14 requirement [7]. Yielding in the first-floor columns is permitted at the supports. However, to sufficiently engage beams in dissipating energy through plastic deformation, it is required that column hinging at the supports appear later than in the beams framing with the column top end. Following the yielding in the first-floor beams, the inflection point of a first-floor column moves up and is assumed to be located approximately at 3/4 of column height, from which the needed flexural strength of a column at the support is determined. The code-specified maximum and minimum reinforcement ratios of a section also need to be satisfied. Moreover, based on ACI 318-14 [7], the positive moment strength of a beam shall be at least half of the negative moment resistance.

Despite all these design requirements, the total number of design variables for the entire system still far exceeds the two decision variables \( \Omega_c \) and \( W_s \) in Eq. (1). This study seeks to reduce the number of design variables to two, one for flexural stiffness and another for strength. It is required that the aspect ratio of cross sectional dimension and the ratio of flexural strength among different members determined from the initial design are maintained during optimization. Two normalized parameters are defined and uniformly applied to all components at a step of optimization. A relative elastic stiffness factor \( \lambda \) is defined as the ratio of effective moment of inertia of a section to that in the initial design. A relative strength factor \( \alpha \) is defined as

\[
\alpha = \frac{M_n - M_G}{M_{n,0} - M_G}
\]

where \( M_n \) is flexural strength of a member, \( M_{n,0} \) is flexural strength determined from the initial design, and \( M_G \) is bending moment caused only by the gravity loads considered in seismic design. \( \lambda = 1 \) and \( \alpha = 1 \) correspond to the initial design. The parameter \( \lambda \) correlates concrete volume \( \Omega_c \). The parameter \( \alpha \) can be translated into the consumption of flexural reinforcement \( W_s \) and indicates the degree of reducing seismic load from that considered in the initial design. Because a RC frame also needs to be designed for pure gravity loading condition, a relative
strength factor \( \alpha_G \) is defined as a lower bound of \( \alpha \). \( \alpha_G \) is evaluated using Eq. (2) by replacing \( M_n \) with \( M_{G,0} \), the bending moment demand in the design for pure gravity loading.

**Determination of Feasible Region Boundary**

**Overview**

The feasible region boundary for solving Eq. (1) is determined first in the \( \lambda - \alpha \) domain. Any point situated on the boundary of feasible region shown in Fig. 1(b) can be interpreted as the minimal \( \lambda \) satisfying performance criteria at a given relative strength factor \( \alpha \). A series of discrete \( \alpha \) values ranging from \( \alpha_G \) to 1 are selected. For each \( \alpha \), the flexural strength of a member, \( M_n \), is modified from the initial design based on Eq. (2). At this strength level, \( \lambda \) is minimized for each level of earthquake hazard using a procedure described later; the controlling value of \( \lambda \) gives the optimal \( \lambda \) for the specific \( \alpha \). This procedure is repeated for all selected \( \alpha \) values so that a feasible region boundary in the domain of \( \lambda \) and \( \alpha \) is defined.

When minimizing \( \lambda \) at a given \( \alpha \), the cross-sectional area of RC frame members is reduced. Accordingly, the maximum reinforcement ratio, \( \rho_{\text{max}} \), needs to be considered as an active restraint in addition to limiting inter-story drift ratio and plastic hinge rotation. The flexural design of a section may also be controlled by minimum reinforcement ratio specified in design codes; however, this occurs normally at very few locations in a frame. Thus, this requirement is implemented only after the design optimization for \( \lambda \) has been completed. Other design requirements described previously, such as those related to strong-column/weak-beam and positive flexural strength of a beam section relative to its negative flexural capacity, are not considered as optimization restraints, because they are automatically satisfied once they have been enforced in the initial design. The first-stage optimization problem that minimizes the relative stiffness factor \( \lambda \) at a given relative strength factor \( \alpha \) can then be stated as

\[
\text{Minimize} \quad \lambda \\
\text{Subject to} \quad \forall \gamma_{\lambda} \leq [\gamma], \; \forall \theta_{\lambda} \leq [\theta]
\]  

(3)

where \( \gamma_{\lambda} \) and \( \theta_{\lambda} \) are inter-story drift ratio and beam plastic hinge rotation in the structure where the flexural stiffness of members has been uniformly modified by \( \lambda \) from the initial design. For each pair of \( \lambda \) and \( \alpha \) defining the feasible region boundary, a flexural design is conducted and the needed longitudinal reinforcement at the critical sections are evaluated. The results are then transformed into pairs of \( \Omega_c \) and \( W_s \) constituting a feasible region boundary in the \( \Omega_c-W_s \) domain where the second-stage optimization defined by Eq. (1) is completed.

**Load-Deformation Response due to Modified Flexural Stiffness**

For each selected \( \alpha \) (\( \alpha_G \leq \alpha \leq 1 \)), a pushover analysis is conducted on the RC frame without stiffness modification from the initial design (i.e., \( \lambda = 1 \)). The analysis provides information regarding base shear, roof displacement, inter-story drift ratio, and plastic hinge rotation. The beams and columns are modeled by line elements with plastic hinges at their ends. Following
gravity loading, lateral loads are applied until a target roof displacement is reached. If the variation of inertia force distribution during inelastic response is considered, an adaptive lateral load pattern accounting for the effects of higher modes and member yielding can be used. The response of structure with \( \lambda = 1 \) during nonlinear static analysis consists of a series of events signified by the generation of plastic hinges; however, the system stiffness matrix \( [K] \) is constant between two subsequent hinging. Lateral loading that results in the formation of the \( j \)th plastic hinge is taken as loading step \( j \) designated by a subscript with parenthesis. The incremental displacement \( \{\Delta u\}_{(j)} \) under a lateral load increase of \( \{\Delta F\}_{(j)} \) satisfies

\[
\{\Delta F\}_{(j)} = \{K\}_{(j)} \{\Delta u\}_{(j)}.
\]

Thus, the total displacement \( \{u\}_{(j)} \) at the \( j \)th loading step due to the load of \( \{F\}_{(j)} \) is

\[
\{u\}_{(j)} = \sum_{k=1}^{j} \{\Delta u\}_{(k)} = \sum_{k=1}^{j} [K]^{-1}_{(k)} \{\Delta F\}_{(k)}.
\] (4)

When \( \lambda \neq 1 \) is uniformly applied to all members, the relative flexural stiffness among different members remains unchanged. The stiffness matrix of the new system can therefore be expressed as \( [K_{\lambda}]_{(j)} = \lambda [K]_{(j)} \). Additionally, at a given \( \alpha \), the introduction of \( \lambda \) has no impact on the load increase \( \{\Delta F_{\lambda}\}_{(j)} \) needed to generate a new plastic hinge, i.e., \( \{\Delta F_{\lambda}\}_{(j)} = \{\Delta F\}_{(j)} \). Thus, the displacement of the system with modified stiffness can be derived as

\[
\{u_{\lambda}\}_{(j)} = \sum_{k=1}^{j} [K_{\lambda}]^{-1}_{(k)} \{\Delta F\}_{(k)} = \sum_{k=1}^{j} \lambda [K]^{-1}_{(k)} \{\Delta F\}_{(k)} = \frac{1}{\lambda} \{u\}_{(j)}.
\] (5)

Accordingly, under the same lateral load prior to reaching a collapse mechanism, the roof displacement \( \delta_{\lambda} \), inter-story drift ratio \( \gamma_{\lambda} \), and plastic hinge rotation angle \( \theta_{\lambda} \) of the system modified by \( \lambda \) must satisfy

\[
\frac{\delta_{\lambda}}{\delta} = \frac{\gamma_{\lambda}}{\gamma} = \frac{\theta_{\lambda}}{\theta} = \frac{1}{\lambda}.
\] (6)

where \( \delta, \gamma, \) and \( \theta \) are roof displacement, inter-story drift ratio, and plastic rotation angle for the structure without stiffness modification (\( \lambda = 1 \)). The simple relationship given in Eq. (6) provides great convenience for solving the first-stage optimization problem defined in Eq. (3). As shown in Fig. 2, for each \( \alpha \), once the base shear vs. roof displacement (V-\( \delta \)) response of a frame system without stiffness reduction becomes available, it can be used to directly construct a nonlinear V-\( \delta \) response for the system with modified flexural stiffness based on the value of \( \lambda \) and Eq. (6). Accordingly, there is no need for running extra analyses during the process of searching for optimal \( \lambda \). Thus, the total number of pushover analyses needed to complete the proposed optimal seismic design is the same as the number of \( \alpha \) values (\( \alpha_{\min} \leq \alpha \leq 1 \)) chosen to define a discretized feasible region boundary.
Determination of Optimal Stiffness at a Given Flexural Strength

For each relative strength factor $\alpha$, pushover analysis is conducted to evaluate $\gamma$ and $\theta$ at target roof displacement of the structure without stiffness modification. If neither $[\gamma]$ nor $[\theta]$ of any hazard level is exceeded, $\lambda$ should be decreased according to Eq. (6) until one of the restraining conditions in Eq. (3) is reached. When $\lambda$ is decreased, axial and shear force within plastic hinge region, section dimensions, structural fundamental period, and reinforcement ratio will change simultaneously, leading to decreased plastic hinge rotation capacity, $[\theta]$, and increased target top displacement. Therefore, an iteration approach is employed to modify target top displacement and plastic hinge rotation capacity, $[\theta]$. The iteration approach modifies capacity spectrum by decreasing $\lambda$, as shown in Fig. 3. The target top displacement is then reevaluated based on the new capacity spectrum and new demand curve, as shown in Fig. 3. After that, the internal force and section dimensions recorded at the new target top displacement are used to determine nonlinear deformation capacity and demand. This procedure is repeated until any deformation capacity equals to the corresponding demand.
Specifically, an iteration procedure starts from \( \lambda^{(0)} = 1 \), and a bilinear simplified capacity spectrum is selected to determine target displacement, \( D_{\text{max}}^{(0)} \), and corresponding story drift ratio, \( \gamma^{(0)} \), and plastic rotation angle, \( \theta^{(0)} \). The section dimensions and internal force within plastic hinge region recorded at \( D_{\text{max}}^{(0)} \) are used to determine \( [\theta]^{(0)} \). Because both \( \gamma^{(0)} \) and \( \theta^{(0)} \) are less than \( [\gamma] \) (constant for each performance level) and \( [\theta]^{(0)} \), \( \lambda \) is decreased to \( \lambda^{(1)} \), and this procedure is repeated until either \( \gamma^{(j)} \) or \( \theta^{(j)} \) is equal to \( [\gamma] \) or \( [\theta]^{(j)} \).

**Summary of Optimization Procedure**

Step 1: An initial seismic design (\( \lambda = 1 \) and \( \alpha = 1 \)) is performed using a strength-based design approach. All design requirements described previously, except for the minimum reinforcement ratio requirement, are followed. The aspect ratio of a cross section and the ratio of flexural strength among different members will be maintained during optimizations. Step 2: A series of relative strength factors \( \alpha \) are selected. For each \( \alpha \), the member flexural strengths in the initial design are uniformly reduced, and a nonlinear static analysis is conducted. Seismic hazards are defined using elastic spectral accelerations. Replaceable pushover method is then applied to determine seismic deformation demands. Step 3: For each \( \alpha \), the relative elastic stiffness factor \( \lambda \) is optimized by solving Eq. (3) using the numerical approach presented in this study. The feasible region boundary is defined based on the pairs of \( \alpha \) and minimized \( \lambda \). Step 4: The feasible region boundary determined in Step 3 is converted into that in \( \Omega_{c-W_s} \) domain. The optimal design is obtained by a mathematical programming or graphical approach. Step 5: The flexural design of beams and columns is performed based on the optimal solution. Code-required minimum reinforcement ratio is checked.

**Application of Proposed Optimization Approach**

The suggested optimal design approach is applied to a six-story RC frame building. The building site is assumed as soft rock in southern California, where the mapped short-period and 1-sec spectral accelerations for risk-targeted maximum consider earthquake (MCE\(_R\)) are \( S_s = 1.50 \)g and \( S_1 = 0.60 \)g, respectively. Concrete compressive strength is assumed as 34.5 MPa and steel reinforcement yield strength as 414 MPa. The floors consist of 203 mm thick two-way slabs. A dead load of DL = 5.60 kN/m\(^2\) and a live load of LL = 0.958 kN/m\(^2\) act on the roof, whereas these values are 5.74 kN/m\(^2\) and 2.39 kN/m\(^2\) for all other floors. The height of each floor and length of each span are as 3.66 m and 9.14 m, respectively. The section size is chosen as 559 mm \( \times \) 813 mm (1\(^{\text{st}}\) to 3\(^{\text{rd}}\) floors) and 508 mm \( \times \) 711 mm (4\(^{\text{th}}\) to 6\(^{\text{th}}\) floors) for the beams, and 737 mm \( \times \) 737 mm (1\(^{\text{st}}\) to 3\(^{\text{rd}}\) floors) and 610 mm \( \times \) 610 mm (4\(^{\text{th}}\) to 6\(^{\text{th}}\) floors) for the columns. This initial design is optimized with target performance of IO, LS, and CP under occasional, rare, and very rare earthquakes, respectively. The occasional and rare earthquakes are defined as the events having 50% and 10% probability of exceedance in 50 years. The very rare earthquake is defined as MCE\(_R\) per ASCE 7-10 [8]. [\( \gamma \)] and [\( \theta \)] are defined based on ASCE 41-06 [2] and ASCE 41-13 [1], respectively. OpenSees [9] is used as a simulation platform.

Relative strength factors from \( \alpha = 1 \) to 0.28 are selected. The minimum \( \lambda \) for each \( \alpha \) are obtained according to optimization procedure described previously, as shown in Fig. 4. It is seen
that the optimal $\lambda$ is controlled by either inter-story drift ratio limit for LS under 10%/50-yr hazard if $0.5 \leq \alpha \leq 1$ or plastic-hinge rotation limit for IO under 50%/50-yr hazard if $\alpha < 0.5$. If no strength reduction is considered ($\alpha = 1$), the optimal relative stiffness factor is determined as $\lambda = 0.51$, which can be translated into a 15.5% reduction in section dimension for all the beams and columns, as compared with the initial strength-based design. At $\alpha = 0.28$ (a 72% reduction of design seismic load), $\lambda = 0.88$, corresponding to member section size almost identical to the initial design. This result indicates that, even if the code-specified earthquake load is significantly reduced in a design, the seismic performance of the 6-story building likely still satisfies the chosen performance criteria. Fig. 4 also indicates that, if inter-story drift ratio is not taken as a constraint, the optimization would be controlled by plastic hinge rotation for LS under 10%/50-yr hazard and reducing section size increases needed member flexural strength. In other words, flexural strength and stiffness are not independent in the optimal solution.

![Feasible Region](image)

**Figure 4. Feasible region in $\alpha$–$\lambda$ domain**

For each pair of $\alpha$ and $\lambda$ on the feasible region boundary shown in Fig. 4, the needed section size and amount of flexural reinforcement for beams and columns are converted into $\Omega_c$–$W_s$ domain. The unit costs of concrete and reinforcement, including material and labor cost are calculated based on BNi Building News [10] and RS Means [11]. It is found that the optimal design corresponds to a 30% reduction in seismic design loads and a 14% reduction in section dimension from the initial strength-based design. The cost of an interior frame (including the beams in the transverse direction) with the initial flexural design and that with the optimal performance-based design are evaluated and compared in Table 1. The optimal design leads to a cost reduction of 18.9%, 22.4%, and 21.1% for the material, labor, and material plus labor, respectively.
### Table 1. Comparison of cost for the initial and optimal designs (unit: $)

<table>
<thead>
<tr>
<th></th>
<th>Initial Design</th>
<th>Optimal Design</th>
<th>Cost Reduction</th>
<th></th>
<th>Initial Design</th>
<th>Optimal Design</th>
<th>Cost Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>37,497</td>
<td>27,977</td>
<td>25.4 %</td>
<td>Reinforcement</td>
<td>17,305</td>
<td>16,441</td>
<td>4.99 %</td>
</tr>
<tr>
<td>Labor</td>
<td>86,098</td>
<td>66,805</td>
<td>22.4 %</td>
<td>Total cost</td>
<td>140,900</td>
<td>111,223</td>
<td>21.1 %</td>
</tr>
</tbody>
</table>

### Conclusions

Taking both inter-story drift and plastic hinge rotation as constraints, this study develops a framework of optimal PBSD for multi-story RC moment frames. The proposed approach has two key features: a two-stage optimization for defining feasible region and solving the optimization problem, and the incorporation of pushover method, which estimates peak dynamic response using nonlinear static analysis, into optimization process. The proposed optimization approach is applied to the design of a six-story moment frame that employs the drift and plastic hinge rotation limits recommended by ASCE 41-06 and ASCE 41-13. The optimal design result is controlled by inter-story drift limit for rare earthquakes (10% exceedance rate in 50 years). Compared with the conventional strength-based design following ASCE 7-10, the optimal design leads to 21% cost reduction.

### References