



## SEISMIC DRIFT PERFORMANCE-BASED DESIGN OPTIMIZATION OF REINFORCED CONCRETE BUILDINGS

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### SUMMARY

Performance-based design is a modern approach to seismic engineering, in which the design aim is to deliver a structure capable of meeting certain predictable performance objectives under different levels of earthquake motions. Performance-based design using nonlinear pushover analysis, which generally involves tedious and intensive computational effort, is a highly iterative process to meet designer-specified and code requirements. This paper presents an effective computer-based technique that incorporates pushover analysis together with numerical optimization procedures to automate the pushover drift performance design of reinforced concrete (RC) buildings. Steel reinforcement, as compared with concrete materials, appears to be the more cost-effective material that can be effectively used to control drift beyond the occurrence of first yielding and to provide the required ductility of RC building frameworks. In this study, steel reinforcement ratios are taken as design variables during the design optimization process. Using the principle of virtual work, the nonlinear inelastic seismic drift responses generated by the pushover analysis can be explicitly expressed in terms of steel reinforcement design variables. An Optimality Criteria technique is presented in this paper to solve the explicit performance-based seismic design optimization problem for RC buildings. One building frame example is presented to illustrate the effectiveness and practicality of the proposed optimal design method.

### INTRODUCTION

The concept of performance-based design has become the future direction of seismic design codes [1-3]. In the newly developed performance-based seismic design approach, nonlinear analysis procedures become important in identifying the patterns and levels of damage to assess a structure's inelastic behavior and to understand the modes of failure of the structure during severe seismic events. Pushover analysis is a simplified, static, nonlinear procedure in which a predefined pattern of earthquake loads is applied incrementally to framework structures until a plastic collapse mechanism is reached. This analysis method generally adopts a lumped-plasticity approach that tracks the spread of inelasticity through the formation of nonlinear plastic hinges at the frame element's ends during the incremental loading process. In general, the determination of the satisfactory performance response that fulfills both the system level

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response and element level response requires a highly iterative trial-and-error design procedure even with the aid of today's engineering computer software.

It has been recognized that the interstory drift performance of a multistory building is an important measure of structural and non-structural damage of the building under various levels of earthquake motion [4]. In performance-based design, interstory drift performance has become a principal design consideration [1,2]. The system performance levels of a multistory building are evaluated based on the interstory drift values along the height of the building under different levels of earthquake motion [5]. The control of interstory drift can also be considered as a means to provide uniform ductility over all stories of the building. A large story drift may result in the occurrence of a weak story that may cause catastrophic building collapse in a seismic event. Therefore, uniform story ductility over all stories for a multistory building is usually desired in seismic design [6].

Although lateral drift performance is a principal concern in the seismic design of structures, economically designing elements of building structures for various levels of elastic and inelastic lateral drift performance under multiple levels of earthquake load is generally a rather difficult and challenging task. Lateral drift design requires the consideration of a proper distribution of the stiffness of all structural elements and, in a severe seismic event, also the occurrence and redistribution of plasticity in the structural elements. Structural engineers are faced with the problem of efficiently proportioning structural materials throughout the building to limit the inelastic seismic drift responses of a structure. Due to the lack of an automated optimization technique, performance-based seismic drift design is usually carried out by trial-and-error methods based on intuition and experience. Chan [7] developed an efficient computer-based optimization technique for lateral stiffness design of tall buildings. Although this research has resulted in actual applications to numerous notable tall building projects in Hong Kong, it should be noted that the research has been primarily focused on the elastic wind drift performance of tall buildings. Much effort is still needed to extend the current optimization technique to inelastic seismic design of multi-story buildings.

This paper presents an effective optimization technique for the inelastic drift performance design of RC building frames under pushover loading. Attempts have been made to automate the performance-based seismic design of RC buildings using an optimization procedure. The quantities of steel reinforcement, the only effective material that provides ductility to RC building frameworks, are considered as design variables in the inelastic seismic drift optimization. With careful tracking of the formation of plastic hinges, the pushover drift can be explicitly expressed in terms of the sizing variables using the principle of virtual work. The optimization methodology for the solution of the nonlinear seismic drift design of buildings is fundamentally based on an Optimality Criteria (OC) approach. A ten-story, two-bay planar frame building is then presented to illustrate the details of the OC optimization method for inelastic seismic drift performance-based design.

## **OPTIMAL INELASTIC SEISMIC DESIGN PROBLEM**

### **Implicit Design Optimization Problem**

In seismic design, it is commonly assumed that a building behaves linear-elastically under minor earthquakes and may respond nonlinear-inelastically when subjected to moderate and severe earthquakes. Under such an assumption, the entire design optimization process can therefore be decomposed into two phases [8, 9]. In the first phase, the structural concrete cost is minimized subject to elastic drift responses under minor earthquake loading using elastic response spectrum analysis. In this phase, concrete member sizes are considered as design variables since the concrete material plays a more dominant role in improving the elastic drift performance of the building. Once the optimal structural member sizes are determined at the end of the first phase of the optimization, the steel reinforcement quantities can then be considered as design variables in the second phase. In controlling the inelastic drift responses, steel reinforcement is the only effective material that provides ductility to an RC building structure beyond first yielding. In this second design phase, the member sizes are kept unchanged and the cost of the steel

reinforcement is minimized subject to design constraints on inelastic interstory drift produced by the nonlinear pushover analysis. The emphasis of this paper is on the second phase of the design optimization, the inelastic seismic drift design optimization. The details of the first phase elastic seismic drift design optimization can be found in the work of Zou [9].

For an RC building having  $i=1, 2, \dots, N_i$  members and  $2N_i$  plastic hinges (assuming one hinge at each end of a member), the tension steel reinforcement ratio,  $\rho_i$ , and the compression steel reinforcement ratio,  $\rho'_i$ , for a rectangular cross section are taken as design variables in the design optimization, whereas the member sizes,  $B_i$  (width) and  $D_i$  (depth), are fixed. If the topology of a building's structural system is predefined, the steel reinforcement cost of the RC framework is minimized as

$$\text{Minimize:} \quad \text{steel cost} = \sum_{i=1}^{N_i} w_{si} (L_{si} \rho_i + L'_{si} \rho'_i) \quad (1)$$

where  $w_{si}$  is the cost coefficient for steel reinforcements; and  $L_{si}$  and  $L'_{si}$  are respectively the lengths of the tension and compression steel reinforcements for member  $i$ . Only the longitudinal flexural reinforcement of member sections is considered as design variables in this study, while the transverse shear reinforcement is considered invariant under the assumption that adequate shear capacity strength is provided for each member.

In the performance-based design, it is necessary to check the “capacity” of a structure against the “demand” of an earthquake at the performance point which is the intersection of the pushover capacity and demand spectrum curves. In this study, the interstory drift responses of a building, generated by a specified earthquake demand, are checked against appropriate limits corresponding to a given performance level. Namely, for a multistory building structure, the interstory drift ratio caused by pushover loading should comply with the following requirement:

$$\frac{\Delta u_j}{h_j} = \frac{u_j - u_{j-1}}{h_j} \leq d_j^U \quad (2)$$

where  $\Delta u_j$  is the interstory drift of the  $j$ th story;  $u_j$  and  $u_{j-1}$  are the respective story displacement of two adjacent  $j$  and  $j-1$  floor levels;  $h_j$  is the  $j$ th story height; and  $d_j^U$  is the specified interstory drift ratio limit representing the damage threshold for the  $j$ th story.

Besides the considerations of the interstory drift responses, local element responses such as the sectional plastic rotation and member strength at the performance point must also be checked against certain acceptability limits. The plastic rotation,  $\theta_{ph}$ , at the  $h$ th end section of a member (where subscript  $h$  represents one end of a member and  $h=1, 2$ ) should be checked as

$$\theta_{ph} \leq \theta_p^U \quad (3)$$

where  $\theta_p^U$  is the plastic rotation limit corresponding to a specific performance level. Once the designer determines the performance levels of the structure (e.g., Immediate Occupancy, Life Safety, Collapse Prevention), the limiting values of  $\theta_p^U$  for all members is then determined. Unless specific design criteria are stated, otherwise FEMA-273 [3] and ATC-40 [2] provide guidelines for estimating the limiting values of plastic rotation of a flexural member for various levels of performance criteria of an RC frame. In practical multistory building structures, excessive number of design constraints may create enormous increases in computational effort. In order to reduce the practical building design problem to a manageable size, the strength design of each member is not considered explicitly as a design constraint; rather, the strength-based steel reinforcement ratios in accordance with code specifications are first calculated and these values are then taken as the lower size bound for each member in the inelastic seismic drift design optimization.

In addition to the design performance constraints on the system-level story drift and element-level sectional plastic rotation, the steel reinforcement variables are imposed within the minimum and maximum steel reinforcement ratios as

$$\rho_i^L \leq \rho_i \leq \rho_i^U; \quad \rho_i'^L \leq \rho_i' \leq \rho_i'^U \quad (4a, b)$$

where the superscripts  $L$  and  $U$  denote the minimum and maximum limits of the design variables,  $\rho_i$  and  $\rho_i'$ .

In order to facilitate a numerical solution of the drift design problem, it is necessary that the implicit story drift constraint (Eq. 2) and the plastic rotation constraint (Eq. 3) be expressed explicitly in terms of the design variables,  $\rho_i$  and  $\rho_i'$ .

### Explicit Drift Formulation

Based on the internal element forces and moments of the structure obtained from the pushover analysis at the performance point, the principle of virtual work can be employed to express the pushover displacement. The pushover story displacement,  $u_j$ , at the performance point includes the virtual work,  $u_{j,memb}$ , produced by the structural members and the virtual work,  $u_{j,hinge}$ , generated by the plastic hinges. That is,

$$u_j = u_{j,memb} + u_{j,hinge} \quad (5)$$

in which

$$u_{j,memb}(B_i, D_i) = \sum_{i=1}^{N_i} \left( \frac{C_{0ij}}{B_i D_i} + \frac{C_{1ij}}{B_i D_i^3} + \frac{C_{2ij}}{B_i^3 D_i} \right)_i \quad (6)$$

$$u_{j,hinge} = \sum_{i=1}^{N_i} \left[ \sum_{h=1}^2 m_{pjh}^0 \theta_{ph} \right]_i \quad (7)$$

In Eq. (6), the displacement,  $u_{j,memb}$ , is expressed in terms of width ( $B_i$ ) and depth ( $D_i$ ) [7, 9]. During the inelastic drift design optimization process,  $u_{j,memb}$  is kept unchanged since  $B_i$  and  $D_i$  of each member section are fixed. The emphasis here is on the displacement,  $u_{j,hinge}$ , caused by the formation of the plastic hinges. In Eq. (7),  $m_{pjh}^0$  is the virtual end moment at the location of the  $h$ th hinge of a member;  $\theta_{ph}$  is the actual plastic rotation experienced by the  $h$ th plastic hinge, which is equal to zero when no plastic hinge is found. As shown in Fig. 1, the behavior of a plastic hinge is modeled as a bilinear curve: the elastic segment,  $AB$ , and the hardening segment,  $BC$ , where Point A corresponds to the unloaded condition, Point B is the first yield moment point, Point C is the ultimate moment capacity, which generally corresponds to the structural stability performance level in ATC-40 [2]. Based on the line segments A-B-C, the plastic rotation,  $\theta_p$ , can be given as follows

$$\theta_p = \frac{M - M_y}{M_u - M_y} \theta_p^U \leq \theta_p^U \quad (8)$$

where  $\theta_p^U$  is the ultimate plastic rotation which can be established based on experimental tests or can be obtained directly from design guidelines such as the ATC-40 [2];  $M$  is the applied moment at the location of the plastic hinge;  $M_y$  is the bending moment at the first yielding of the tensile steel; and  $M_u$  is the ultimate moment resistance. Given the quantity of the steel reinforcement used in a concrete section, the values of  $M_y$  and  $M_u$  can then be determined. For simplicity,  $M_u$  can be approximately related to  $M_y$

as  $M_u = 1.1M_y$  [2].

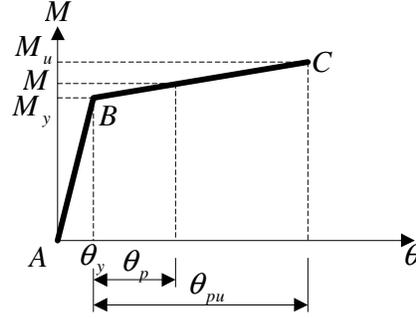
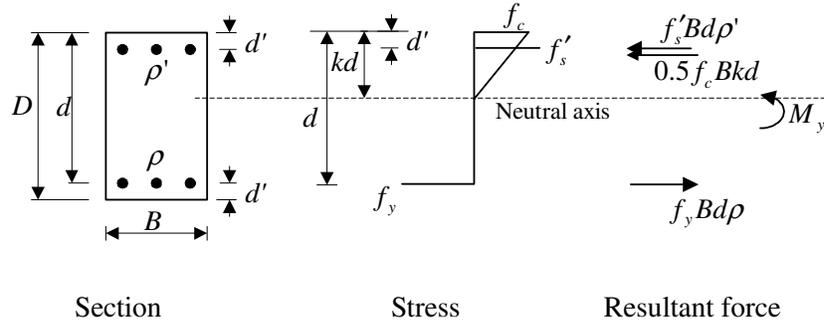


Fig. 1. Moment-rotation curve

For the explicit problem formulation, it is necessary that the plastic rotation,  $\theta_p$ , be accurately expressed in terms of the design variables (i.e.,  $\rho$  and  $\rho'$ ). Furthermore, a good formulation should reflect accurately the change in the plastic rotation,  $\theta_p$ , due to a change in the design variables during the optimization resizing process. In other words, any change in the design variables,  $\rho$  and  $\rho'$ , during the inelastic optimization process requires a corresponding update on the values of  $M$  and  $M_y$ .



Section Stress Resultant force  
Fig. 2. Double reinforced member section at first yield

In pushover analysis, moment hinges are assumed and are generally assigned to the two ends of each beam or column. By the force equilibrium shown in Fig. 2, where  $f_c$  is the stress at the extreme compression concrete fiber,  $f'_s$  is the stress in the compression steel,  $f_y$  is the yield strength of the tension steel, and  $d$  is the effective depth, which is equal to the distance from the extreme compression fiber to the centroid of the tension steel,  $M_y$  for a moment hinge can be expressed in terms of design variables,  $\rho$  and  $\rho'$ , as

$$M_y = 0.5 f_c B k d \left( \frac{k d}{3} - d' \right) + f_y B d (d - d') \rho \quad (9)$$

where  $k$  is the neutral axis depth factor at the first yield and it is given as

$$k = \sqrt{(\rho + \rho')^2 n_{sc}^2 + 2(\rho + \rho') \frac{d'}{d} n_{sc} - (\rho + \rho') n_{sc}} \quad (10)$$

in which  $n_{sc} = \frac{E_s}{E_c}$  where  $E_c$  and  $E_s$  are the moduli of elasticity of the concrete and of the steel, respectively. To take into account the change in  $\theta_p$  due to a change in  $\rho$  and  $\rho'$  while maintaining an instantaneously fixed value of  $M$ , a second-order Taylor series approximation for evaluating the value of  $\theta_p$  is given as

$$\theta_p(\rho) = \theta_p \Big|_{\rho=\rho^0} + \frac{\partial \theta_p}{\partial \rho} \Big|_{\rho=\rho^0} (\rho - \rho^0) + \frac{1}{2} \frac{\partial^2 \theta_p}{\partial \rho^2} \Big|_{\rho=\rho^0} (\rho - \rho^0)^2 \quad (11)$$

where the tension steel ratio,  $\rho$ , is considered as the major design variable; for simplicity, the compression steel ratio,  $\rho'$ , is assumed to be linearly related to  $\rho$  for beams and to be the same as  $\rho$  for columns. Given the explicit expression of  $M_y$  as a function of  $\rho$  from Eq. (9), the gradient,  $\frac{\partial \theta_p}{\partial \rho}$ , and the second-order term,  $\frac{\partial^2 \theta_p}{\partial \rho^2}$ , can be analytically calculated from Eq. (8).

By substituting the explicit plastic rotation,  $\theta_p(\rho)$ , given in Eq. (11) into Eq. (7), the pushover displacement,  $u_j$ , in Eq. (5) can also be explicitly expressed in terms of the design variable,  $\rho_i$ .

### Plastic Rotation Constraint and Sizing Constraint

In this design optimization, when the plastic rotation,  $\theta_p$ , is to be modified with changes in the design variable,  $\rho_i$ , it is necessary to make sure that  $\theta_p$  does not exceed the specified threshold of plastic rotation,  $\theta_p^U$ , for each specified performance level. Moreover, in order to prevent drastic changes in the internal element force and moment redistribution due to the changes in the design variables resulting in fluctuation of solution convergence during the pushover reanalysis and design optimization processes, it is necessary that each plastic hinge remain plastic once it appears during the resizing iteration of the design variables. Furthermore, to maintain the accuracy of the Taylor approximation of the pushover displacement in Eq. (11), it is necessary to ensure that the variation of  $\rho_i$  for the members with plastic hinges be restricted within a relatively small range.

It is found from Fig. 1 that, in order to maintain the relationship of  $0 \leq \theta_p \leq \theta_p^U$ , the internal moment,  $M$ , leading to the occurrence of a plastic hinge must satisfy the following condition:

$$M_y \leq M \leq M_u \quad (12)$$

As a result, based on Eq. (12), the lower and upper bounds of  $\rho_i$  for each plastic hinge can be instantaneously established during the OC iterative resizing process. It should be noted that the proper establishment of the lower and upper bounds of  $\rho_i$  not only limits the changes in the steel reinforcement design variables, but also it satisfies the local performance-based constraints on the control of the local plastic rotation at the ends of members.

### Explicit Design Problem Formulation

Upon establishing the explicit inelastic drift formulation, the optimization problem of minimizing the steel construction cost of a multistory RC building can be explicitly written in terms of the design variable,  $\rho_i$ , as

$$\text{Minimize:} \quad F(\rho_i) = \sum_{i=1}^{N_i} w'_{si} \rho_i \quad (13)$$

subject to:

$$g_j(\rho_i) = \frac{1}{h_j} \left[ \Delta u_j \Big|_{\rho_i=\rho_i^0} + \sum_{i=1}^{N_i} \alpha_{1i} (\rho_i - \rho_i^0) + \frac{1}{2} \sum_{i=1}^{N_i} \alpha_{2i} (\rho_i - \rho_i^0)^2 \right] \leq d_j^U \quad (j=1, 2, \dots, N_j) \quad (14)$$

$$\rho_i^L \leq \rho_i \leq \rho_i^U \quad (i=1, 2, \dots, N_i) \quad (15)$$

where

$$\alpha_{1i} = \frac{\partial \Delta u_j}{\partial \rho_i} \Big|_{\rho_i=\rho_i^0} = \left[ \sum_{h=1}^2 m_{jh}^0 \frac{\partial \Delta \theta_{ph}}{\partial \rho} \Big|_{\rho_i=\rho_i^0} \right]_i \quad (16a)$$

$$\alpha_{2i} = \frac{\partial^2 \Delta u_j}{\partial \rho_i^2} \Big|_{\rho_i=\rho_i^0} = \left[ \sum_{h=1}^2 m_{jh}^0 \frac{\partial^2 \Delta \theta_{ph}}{\partial \rho^2} \Big|_{\rho_i=\rho_i^0} \right]_i \quad (16b)$$

In Eq. (13),  $w'_{si}$  is the cost coefficient for the steel reinforcement,  $\rho_i$ . Eq. (14) defines the set of seismic interstory drift performance constraints under specified earthquake ground motions. Eq. (15) defines the sizing constraints for the steel reinforcement, where  $\rho_i^L$  and  $\rho_i^U$  correspond to the lower and upper size bounds specified for the tensile steel reinforcement variable,  $\rho_i$ , and they should be updated after each nonlinear pushover analysis.

Once the design optimization problem is explicitly expressed in terms of design variables, the next task is to adopt a suitable method for solving the problem. The OC approach is proposed to solve the explicit inelastic drift optimization problem Eqs. (13)-(15). When using the OC technique, a set of necessary optimality conditions for the design are derived and then a recursive algorithm is applied to resize the structure to satisfy the optimality conditions and thus indirectly optimize the structure. Further details of the OC technique can be found in the reference by Zou [9].

### Initial Preprocessor

Although the OC method does not impose any restrictions on the initial values of the design variables, the rate of convergence of the OC process depends on the initial design values. To speed up the convergence of the OC process, it is important to begin with a reasonably good starting design. One effective scaling approach was proposed by Chan [10]. In such an approach, the design optimization with a single drift constraint is first considered and a simple "closed form" solution for the problem is derived analytically and exploited as an initial preprocessor for the iterative OC process. The advantage of this approach is that a reasonable initial design can be quickly established based on a representative single drift constraint. Experience indicates that the initial preprocessor can generally lead to steady and rapid solution convergence of the multiple inelastic drift optimization problem.

## OVERALL DESIGN OPTIMIZATION PROCEDURE

The overall design optimization procedure for limiting lateral elastic and inelastic drifts of a reinforced concrete building structure is listed as follows.

- (1) Establish an initial design with optimal member sizes, which can be obtained from the elastic seismic design optimization by minimizing the concrete cost of an RC structure subjected to a minor earthquake loading using the elastic response spectrum analysis method [9].
- (2) Determine the design spectrum, corresponding to a severe earthquake event, that will be used in the nonlinear pushover analysis.
- (3) Conduct a static virtual load analysis to obtain the member internal forces that will be used in formulating inelastic drift responses by employing the principle of virtual work.

- (4) Based on the optimal member size, determine the minimum and maximum size bounds of the steel reinforcement ratios,  $\rho_i$  and  $\rho'_i$ , in accordance with the strength-based code requirements.
- (5) Apply the initial preprocessor based on a representative single drift constraint to establish a reasonable starting set of steel reinforcement design variables for the multiple drift constrained optimization.
- (6) Carry out the nonlinear pushover analysis using commercially available software to determine the performance point of the structure and the associated inelastic drift responses of the structure at the performance point.
- (7) Track down the locations of the plastic hinges, establish the instantaneous lower and upper bound move limits of  $\rho_i$  for those members with plastic hinges based on Eq. (12) and determine the values of the first-order and second-order derivatives of the drift responses using Eqs. (16a) and (16b).
- (8) Establish the explicit interstory drift constraints using a second-order Taylor series approximation and formulate the explicit design problem, Eqs. (13)-(15).
- (9) Apply the recursive OC optimization algorithm to resize all steel reinforcement design variables and to identify the active inelastic drift constraints.
- (10) Check convergence of the steel cost and the inelastic drift performance of the structure. Terminate with the optimum design if the solution convergence is found; otherwise, return to Step 6.

### ILLUSTRATIVE EXAMPLE

A ten-story, two-bay planar frame is used to illustrate the proposed optimal design method. The geometry of the example is given in Fig. 3. Concrete with the cylinder strength of 20MPa and steel reinforcement with the yield strength,  $f_y$ , of 335MPa are used for all members. The loads considered in the pushover analysis are lateral seismic loads and vertical gravity loads. While the lateral loads are incrementally applied, the gravity loads are maintained to be unchanged during the nonlinear pushover analysis process. A uniformly distributed gravity load of 30kN/m is to be applied to the beams of each story.

Initial member sizes of the framework used to commence the inelastic design optimization are shown in Table 3. Initial steel reinforcement ratios are first calculated based on the strength requirements of the RC members in accordance with the Chinese seismic design code [11]. Such strength-based reinforcement ratios are taken initially as the lower bounds for the inelastic design optimization. The upper size bounds of the steel reinforcement ratios are assumed to be 6.0% for columns and 4.0% for beams. For simplicity, symmetrical arrangement of the steel reinforcement of each member is assumed such that  $\rho_i = \rho'_i$ . Flexural moment hinges are assigned to the end locations of the beams and columns and the ultimate plastic hinge rotation,  $\theta_p^U$ , is assumed to be 0.02 radian

The 5% damped design spectrum with an initial peak acceleration of 1.4g according to the Chinese seismic design code [11] is modified by the spectral reduction method in the pushover analysis of this example. A typical unit construction cost of the steel reinforcement (including the costs of the steel material and the labor) of US\$950/tonne is assumed. Interstory drift constraints are considered with an assumed allowable interstory drift ratio limit of 1/100. The initial preprocessor, in which analytical optimization with the top displacement constraint alone is considered, is applied before the multiple inelastic interstory drift constrained optimization is invoked. The design process is deemed to converge when the difference in the structure costs for two successive design cycles is within 0.5% and when the difference between the active interstory drift value and its allowable limit at the performance point is within 0.5%.

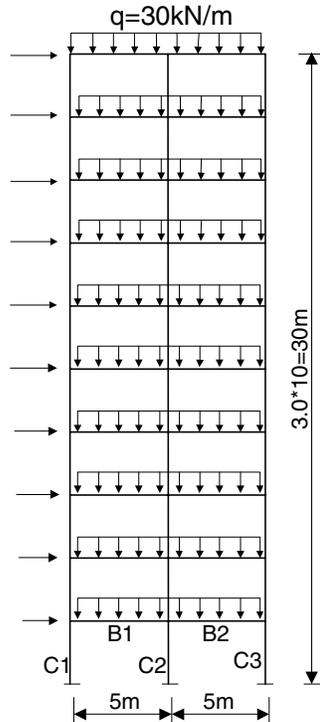


Fig. 3 A Ten-Story, Two-Bay Frame

The optimal design history of the example is presented in Fig. 4. The initial preprocessing is first completed and the multiple inelastic interstory drift constrained optimization is then commenced. It is found that there is a relatively large increase of 14% in the steel cost from the initial US\$1693 to US\$1931 after the initial preprocessing with only the top displacement constraint. However, the optimal design process with the multiple drift constraint converges slowly but steadily within 11 design cycles, with only a slight difference of 2% in the steel cost from US\$1931 to the final US\$1978. Relatively slow, but steady, convergence is found due to the need for maintaining a small change in the steel reinforcement ratios during the inelastic design optimization process.

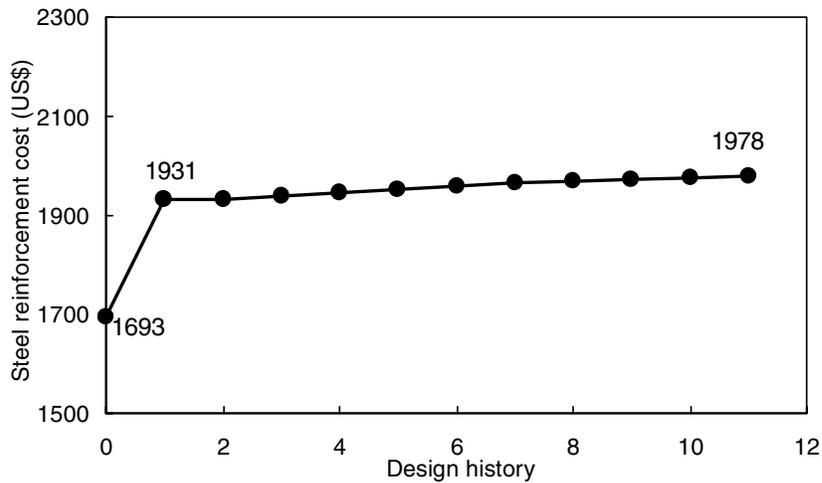


Fig. 4 Design History of Steel Reinforcement Costs

Table 1 Initial and Final Steel Reinforcement Ratios

Element type	Story level	Member group	Initial member sizes		Steel ratios	
			Width (mm)	Depth (mm)	Initial (%)	Optimal (%)
Column	9th~10th	C1,C3	350	350	0.990	1.096
		C2	350	350	1.386	1.386
	8th	C1,C3	350	350	0.849	0.860
		C2	350	475	0.958	0.958
	7th	C1,C3	350	350	1.163	1.163
		C2	350	475	1.092	1.092
	6th	C1,C3	350	400	0.854	0.864
		C2	350	575	0.831	0.831
	5th	C1,C3	350	400	0.969	1.004
		C2	350	575	1.002	1.002
	4th	C1,C3	350	450	0.833	0.873
		C2	350	600	0.739	0.739
	3rd	C1,C3	350	450	0.825	0.876
		C2	350	600	0.857	0.857
	2nd	C1,C3	350	450	1.044	1.124
		C2	350	650	1.225	1.225
	1st	C1,C3	350	450	1.514	1.514
		C2	350	650	1.844	1.844
Beam	9th~10th	B1,B2	200	400	0.800	0.800
	8th	B1,B2	200	450	0.838	0.941
	7th	B1,B2	200	450	0.915	1.320
	6th	B1,B2	250	450	0.897	1.310
	5th	B1,B2	250	450	0.958	1.493
	4th	B1,B2	300	450	0.915	1.457
	3rd	B1,B2	300	450	0.958	1.484
	2nd	B1,B2	300	450	0.982	1.512
	1st	B1,B2	300	450	0.843	1.006

Table 1 presents the initial and optimal steel reinforcement ratios. Initially, the starting design with strength-based steel reinforcement is found to be infeasible in terms of the assumed allowable interstory drift limit. After the optimization, the steel reinforcement ratios of the beams greatly increase particularly in the lower levels of the structure, while those of columns are found with little changes.

Fig. 5 presents the initial and final performance points respectively. The performance point “P1” of the initial structure has a spectral acceleration capacity of 0.068g and a spectral displacement capacity of 0.262m. The optimized structure corresponding to the final performance point “P2” has a spectral acceleration capacity of 0.086g and a spectral displacement of 0.211m. Such a result of shifting the spectral displacement from 0.262m to 0.211m indicates that, for the optimized inelastic frame, the inelastic lateral load resistance has been enhanced through optimal resizing of the steel reinforcement by the OC procedure. Also, shifting the ultimate spectral acceleration capacity from 0.068g to 0.085g indicates that the optimized structure attracts an increase in the seismic loading action and therefore, requires the structure to be stiffened. The OC procedure developed is found to be able to automatically drive from any initial performance point to the final performance point resulting in the minimum cost design.

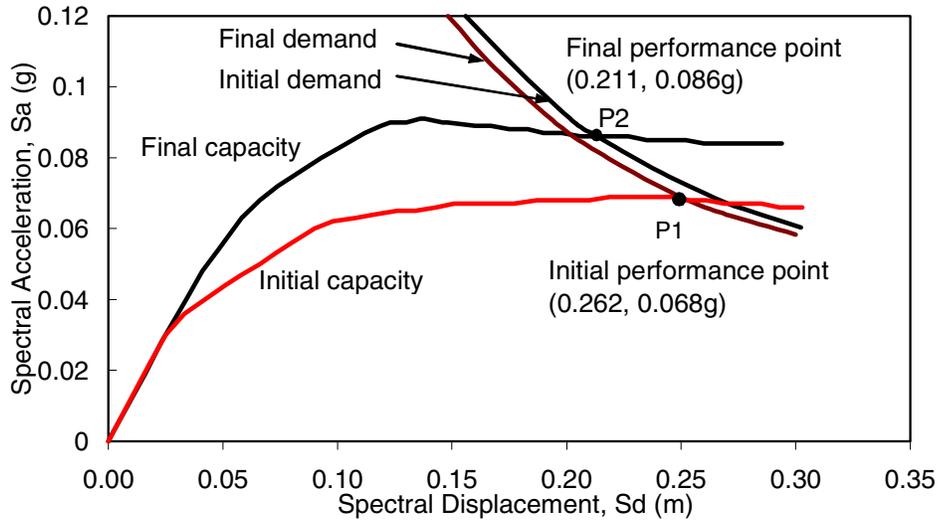


Fig. 5 Comparison of Performance Points

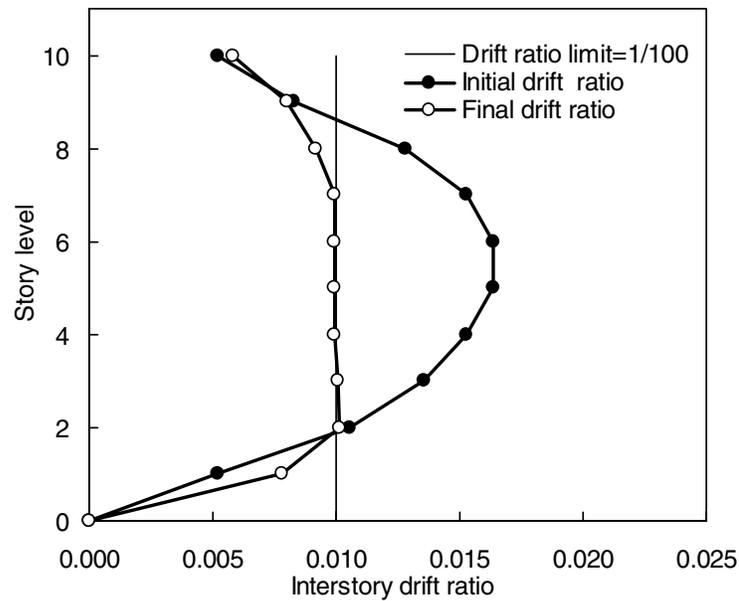
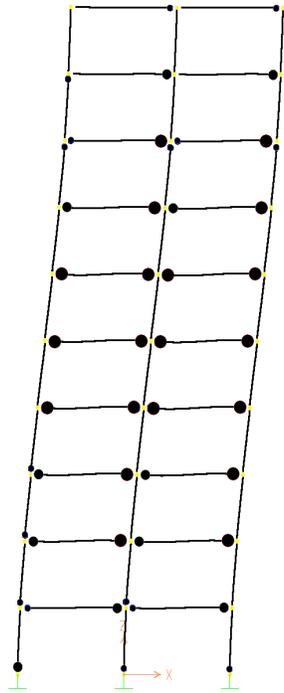
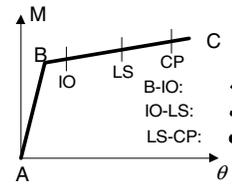


Fig. 6 Initial and Final Interstory Drift Ratios

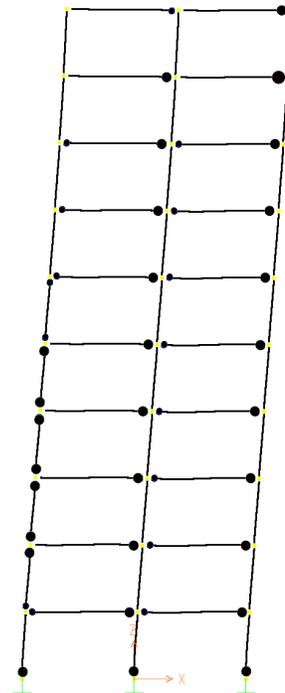
The initial and final interstory drift ratios are shown in Fig. 6. The initial interstory drift constraints at the second through the eighth floors are found to violate substantially the allowable interstory drift ratio limit of 1/100, resulting in the occurrence of the weak stories on these floor levels of the building. However, these pushover interstory drift constraints are found to be close to and within the allowable values after the optimization, indicating that a rather uniform interstory drift distribution over the height of the building has been achieved and the occurrence of weak story has been prevented at the optimum performance point.

	Number of plastic hinges			Total number of plastic hinges
	B-IO	IO-LS	LS-CP	
Initial design	16	11	20	47
Final design	16	28	1	45

(a) Number of Plastic Hinges



(b) Initial Design



(c) Final Design

Fig. 7 Initial and Final Plastic Hinge Distribution

Fig. 7(a) includes a table showing the number of plastic hinges at three different performance states. Figs. 7(b)-7(c) show the initial and final plastic hinge distributions under the pushover loading at the performance point of the structure. No plastic hinge rotation is found to exceed the specified threshold of plastic rotation. As shown in Fig. 7(b), the rotations of twenty plastic hinges of the initial design are found to be located between the LS-CP state. However, after the optimization, most of the plastic hinges are found to be in the B-IO and IO-LS states and only one hinge is in the LS-CP state, as can be observed from the optimized framework in Fig. 7(c). Furthermore, the interstory drifts along the height of the building are also found to be almost all fully constrained at the optimum, resulting in a rather linear deflected profile of the inelastic design. Such a result further indicates that the optimization method developed can automatically resize the steel reinforcements of all members to attain a uniform ductility demand along the height of the multistory building.

## CONCLUSIONS

It has been demonstrated that steel reinforcement plays a significant role in controlling the lateral drift beyond first yielding and in providing ductility to an RC building framework. Using the principle of virtual work and the Taylor series approximation, the inelastic performance-based seismic design problem has been explicitly expressed in terms of the steel reinforcement design variables.

It is demonstrated that the OC design method is able to improve automatically and gradually a performance-based interstory drift design to attain optimal performance. Also, this OC design method developed is able to automatically shift any initial performance point to achieve the final optimal performance point. However, the restrictive move limit imposed on the steel reinforcement design variables is necessary to ensure a smooth and steady convergence of the inelastic drift design process. At optimum, a uniform lateral drift or ductility demand over all stories of the building with the minimum cost is achieved. It is also believed that this optimization methodology provides a powerful computer-based technique for performance-based design of multistory RC building structures. The proposed optimization methodology provides a good basis for more comprehensive performance-based optimization of structures as more accurate nonlinear pushover procedures taking into the higher mode effects are developed and multiple levels of performance criteria and design objectives are to be simultaneously considered.

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