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The Optimal Drift Design Method to Control the Elastic and Inelastic Performance of Steel Moment Frames

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Abstract

This study presents the optimal drift design method to control the elastic and inelastic performance of steel moment frames. This is formulated as a problem that minimizes the lateral displacement at the top of a building satisfying the constraints on the total structural weight and the column-to-beam strength ratios at the joints. This uses the resizing method based on the linear static analysis. The resizing method can increase the stiffness of buildings without the increase of structural weight because of resizing the size of elements based on the displacement participation factor (DPF) calculated by the unit-load method. Simultaneously, this can control the inelastic performance of a building through the constraints on the column-to-beam strength ratios at the joints. The proposed method is demonstrated by application to the steel moment frame example. It is confirmed that the initial stiffness and dissipated energy of the redesigned structure are controlled.

Keywords: steel moment frame, drift design, optimization, column-to-beam strength ratio, resizing.

1 Introduction

Moment frames consist only of columns and beams, and are a simple seismic resisting system with a rigid connection between the columns and beams. They are used widely in many countries for their superior ductility and diversity in architectural design. Since the column-beam connection is rigidly jointed, when an earthquake occurs, the moment frame experiences lateral deformation through the flexural behavior of the columns and beams. Therefore, the flexural strength of the members becomes an important factor in deciding the behavior of the entire building structure [1].

The seismic design of the moment frames considers inter-story drift, as well as the strength of the members [2, 3]. When excessive inter-story drift occurs, it can lead to immense damage to the structural materials and non-structural elements, and can even result in the collapse of the building. Thus, inter-story drift must be checked in the structural design stage. The inter-story drift condition is known to be more dominant in the design element for the structural design of a building than the strength condition of buildings [4, 5]. For this reason, drift design, which controls the displacement of the structure, is performed. However, the inter-story drift of a structure is decided by the influence of various members, so drift designs based on the engineer's experience and intuition can become uneconomical.

To solve such problems, there has been active research regarding the optimum drift design for buildings with a seismic load. Park and Kwon [6] suggested an optimum drift design of a steel moment frame, which used both a response spectrum interpreted model and sensitivity interpreted model. Al-Ansari and Senouci [7] suggested an optimum drift design model for high-rise buildings that used an equivalent static seismic load based on a generalized SDF system. However, most of the researches only considered the performance within the elastic range, and did not examine the inelastic performance. In the case of moment frames, it is effective to increase the size of the beams and decrease the size of the columns in order to increase the stiffness of structures within the elastic range. However, this can induce early collapse, and increases the brittle fracturing possibility of the building. Therefore, in the drift design of the framework in regards to seismic load, it is necessary to consider inelastic performance, as well as elastic performance. Chan and Zou [8] and Zou et al [9] suggested an optimum drift design model of a moment frame using a nonlinear static analysis and the optimality criteria (OC). However, this has a shortcoming that the nonlinear static analysis, which required a lot of computational time, has to be performed repeatedly. In current practice, static elastic analysis, using an equivalent static seismic load, is used for the seismic design of building structures, so such methods where nonlinear static analysis are required to be performed repeatedly, the limits in application arise. Other than these, there is research being performed in regards to the optimum seismic design using heuristic-based optimization methods, such as genetic algorithm (GA) and ant colony optimization (ACO) [10-13]. This requires repeated structural analysis, which generates an excessive amount of computation, so it is difficult to consider it as a realistic solution.

This study suggests an optimum drift design model, which can control the elastic and inelastic performance of the steel moment frame, using linear-static analysis. The proposed method is formulated into a problem that minimizes the lateral displacement at the top of the building, while also satisfying the constraints on the total structural weight and the column-to-beam strength ratio at the joints. This can increase the stiffness of buildings without increasing the structural weight, because the resizing of the size of the elements is based on the displacement participation factor (DPF) calculated by the unit-load method. In other words, the structural weight of each member is efficiently redistributed according to its displacement participation. At the same time, this constrains the column-to-beam strength ratio at the joints, which prevents excessive redistribution of the weight of the columns to the weight of the beams. Hence, the inelastic performance of the structure can be controlled. ANSI/AISC 341-05 [14] suggests the strong column-weak beam concept to retain the ductile behavior of the moment frames. This concept means that the total flexural strength of the columns should be designed to be larger than the total flexural strength of the beams, which results in a plastic hinge occurring in the beams before the columns. Through this, seismic energy is dispersed stably by uniformly inducing the distribution of inter-story displacement. Therefore, constraints on the strength ratio can be applied effectively in retaining the stable dissipated energy capacity of the structures with only the flexural strength of the member and without nonlinear analysis. A three-story steel moment frame was used as an example in this study to verify the proposed method.

2 Formulation of the optimization problem

This study proposes a model that optimizes the stiffness of the steel moment frames using a linear-static analysis. The proposed model controls the stiffness of structures by redesigning the size of each member using a unit load method. Also, the constraint conditions on the column-to-beam strength ratio are utilized to indirectly control the inelastic behavior of the structure. This study uses an objective function which minimizes the top floor displacement using Equation (1), and the constraint conditions regarding the structural weight and column-to-beam strength ratio at the joints using Equation (2)-(4).

$$Minnimize \quad \delta = \sum_{i=1}^{n} \frac{\delta_i}{\beta_i} \tag{1}$$

$$Subject\ to \quad \sum_{i=1}^{n} \rho(\beta_i A_i) l_i \le \gamma \sum_{i=1}^{n} \rho A_i l_i \tag{2}$$

$$\frac{\sum M_j^{pc}}{\sum M_j^{pb}} \ge \alpha_j \quad j = 1\ to\ m \tag{3}$$

$$LB_i \le \beta_i \le UB_i \qquad i = 1\ to\ n \tag{4}$$

Here, δ is the lateral displacement of the top floor. β_i and δ_i are the sectional modification factor and the displacement participation factor regarding the top floor displacement of each $i\ th$ member, respectively. n and ρ are the total number of members of the building, and the material density, respectively. A_i and l_i are the cross-sectional areas and the length of each $i\ th$ member, respectively. γ is the weight control factor of the entire structure, and means the restriction of the weight ratio before and after resizing. α_i, M_j^{pc}, and M_j^{pb} are the limit of the strength ratio of the $j\ th$ joint, and the plastic bending strength of the column and beam, respectively. m is the number of joints. In Equation (4), LB_i and UB_i are each the lower and upper limits regarding the section change ratio of the $i\ th$ member. δ_i is obtained using the unit load method [15].

3 Optimal drift design model

3.1 Method of resizing elements

β_i is a design variable used in formulation, and is used to redesign the cross section of members. In other words, when the above optimization problem is solved, the β_i of each member can be obtained, and when this is multiplied to the A_i of corresponding element, the new cross section $\beta_i\ A_i$ of the *i th* member can be determined. Some sections obtained by this method may not be the sections that are purchased in the market. Therefore, to utilize the proposed method in practice, it is necessary to adjust the newly obtained cross-sectional area to the closest cross-sectional area of the sections commonly used.

For this, this study limits the range of the section list of columns and beams to the W14 type and the W33-36 type respectively, and decides the sections from these. β_i is determined in the direction of decreasing top floor displacement, so there is a tendency to reduce the cross-sectional area of the columns, while increasing the cross-sectional area of the beams. This increases the possibility of brittle fracturing of the structure. Therefore, the cross-sectional area of the column members was set to be the same as $\beta_i\ A_i$ or the smallest cross-sectional area within the column section lists which are larger than $\beta_i\ A_i$ was selected for. On the other hand, the cross-sectional area of the beam members was set to be the same as $\beta_i\ A_i$ or the largest cross-sectional area within the beam section lists which are smaller than $\beta_i\ A_i$ was selected for.

Finally, to consider the constructability of the columns, the cross-sectional areas of the vertically continued column members were compared. If the cross-sectional area of the lower column was smaller than the cross-sectional area of the upper column, the cross-sectional area of the lower column was adjusted to be the cross-section of the upper column.

3.2 Procedure of the optimal drift design model

The entire process of the optimum drift design model, which can control the elastic and inelastic performance of the structures based on the linear static analysis, is as follows:

(1) Assume the initial design.
(2) Obtain the member forces by the design load and unit load applied to the top floor through structural analysis.
(3) Calculate the displacement participation factor of each member.
(4) Set the range of limitation used in Equation (2)-(4).
(5) Solve the problem as shown in Equation (1)-(4) using optimization method to obtain β_i. This study used the fmincon function of Matlab [16].

(6) Decide on the cross-sectional area of each member considering the common DB.
(7) Check the constructability constraints, and adjust the cross-sectional area of the members when they are invalid.
(8) Check the displacement and strength conditions, and when there is a violation, repeat stages (4)-(7).

4 Application

The proposed optimum stiffness model was applied to a three-story steel moment frame [17] as in Figure 1. The external columns, internal columns, and beams were grouped into three design variables, respectively and the cross-sectional areas of the total nine members were controlled to redistribute the structural weight. To verify the performance of the proposed model, the elastic and inelastic performance of the structure, before and after redistribution, was evaluated. This study considered the initial stiffness and the energy dissipation capacity as the elastic and inelastic performance of the structure.

Energy dissipation capacity was calculated as the area of the pushover graph, where the maximum inter-story drift ratio reached 5.0%, which is relevant to the collapse prevention performance standards suggested by FEMA 356 [18]. Midas Gen [19] and OpenSEEs [20] were used in strength design and pushover analysis, respectively.

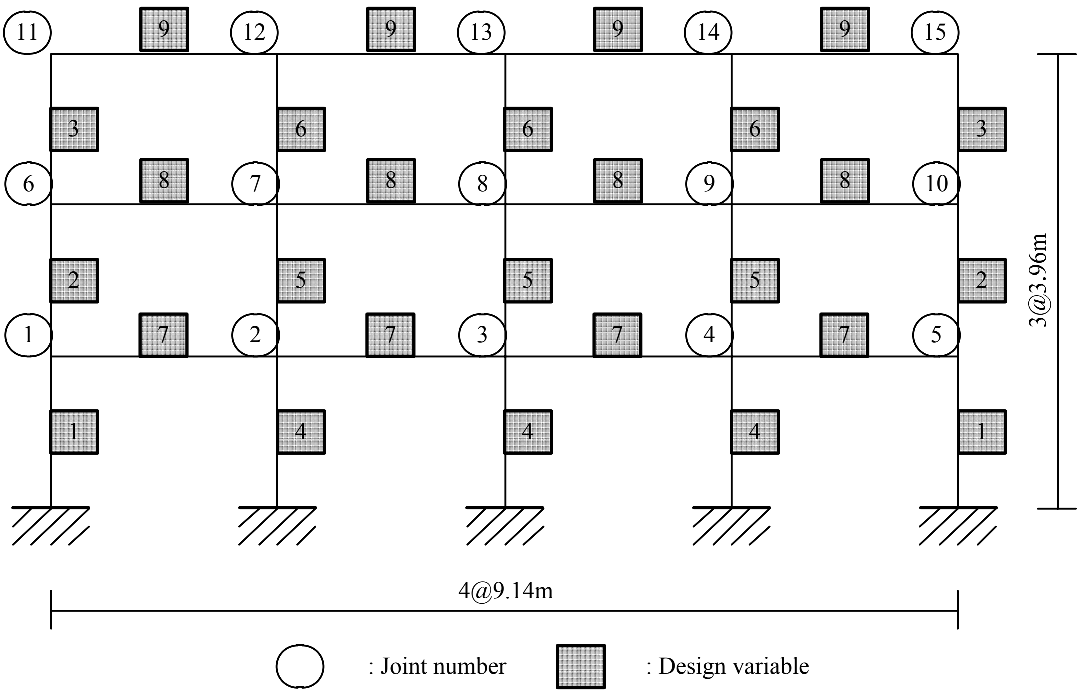

Figure 1: Example elevation

To compare the performance of the proposed model, the initial design before redistribution (Case 1), the design which redistributed using only displacement participation and not considering Equation (3) (Case 2), and the design which redistributed considering all constraint conditions (Case 3) were used as in Table 1.

Design variable	Case 1	Case 2	Case 3
1	W14x257	W14x257	W14x257
2	W14x257	W14x193	W14x193
3	W14x257	W14x193	W14x193
4	W14x311	W14x342	W14x342
5	W14x311	W14x311	W14x311
6	W14x311	W14x257	W14x257
7	W33x118	W30x132	W30x132
8	W30x116	W33x130	W33x118
9	W24x68	W24x68	W24x68

Table 1: Section table of Case 1, 2, and 3

The comparison results were organized in Table 2. Case 2, which did not consider the flexural strength ratio, had increased initial stiffness than Case 1, but the energy dissipation capacity decreased. This is considered to be a result of performing member design according to the displacement participation factor of each member obtained based on the linear-static analysis. On the other hand, Case 3, which additionally considered flexural strength ratio conditions, showed a smaller initial stiffness than Case 2, but showed an increased initial stiffness and energy dissipation capacity than Case 1. This is thought to be from the constraint conditions of the flexural strength ratio effectively being applied in controlling the energy dissipation capacity of the structure.

However, the changes in Case 2 and Case 3, in regards to Case 1, were not large. This is thought to be from the relatively small range of changes in cross-sectional area because of constraining the change range in Equation (4) to be within ±25%.

Performance index	Case 1	Case 2	Case 3
Structural weight (kN)	413.41	405.58	403.47
Initial stiffness (kN/cm)	475.28	498.48	484.38
Total dissipated energy (kN*cm)	36.83	36.55	37.45

Table 2: Performance comparison of Case 1, 2, and 3

5 Conclusions

This paper proposes an optimum drift design model that can control the elastic and inelastic seismic performance of the steel moment frames based on linear-static analysis. This controls the initial stiffness and the inelastic behaviour of the structure using the resizing algorithm, based on the displacement participation factor calculated by the unit-load method, and flexural strength ratio constraints for joints.

The three-storey steel moment frame example was used to verify the proposed model. When the structure was redesigned, with just the displacement participation factor, with no consideration for the flexural strength ratio, the initial stiffness was increased, but inelastic performance, such as energy dissipation capacity decreased. In contrast, when the flexural strength ratio constraints for joints were considered simultaneously, the initial stiffness showed a smaller increase, but inelastic performance was enhanced. In other words, the proposed model was confirmed to be effective in controlling the initial stiffness and energy dissipation capacity of the structure.

This study considered only the initial stiffness and energy dissipation capacity as structural performance of the structure. Therefore, it is necessary to review the performance in regards to more various structural performance indices. Also, the behavioural characteristics of the structure changes according to the flexural strength ratio of each joint, so further research is required.

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