Design of Steel Moment Frames against Collapse

Hyunkoo Kang, Madhuri Thimmappa, and Jinkoo Kim

***Abstract*—** In this study reliability-based design method was developed using energy-balance equation to prevent progressive collapse of steel moment frames caused by sudden loss of a first story column. To consider the uncertainty associated with properties of design variables, a modification factor was computed in such a way that the response of the model structure satisfies a given target reliability index. The statistical properties of the responses were computed using the first order second moment method based on the assumption that the probability distributions of the design variables have the form of normal distribution.

***Index Terms*—**Progressive collapse, Steel structures, Reliability, Nonlinear analysis, Energy-based design.

I. Introduction

Progressive collapse refers to the phenomenon whereby local damage of structural elements caused by abnormal loads results in global collapse of the structure. The progressive collapse-resisting capacity of steel moment frames was assessed [1]. A macro analysis model to investigate the resistance of seismically designed steel braced frames to progressive collapse was applied [2]. An analysis/design integrated system for moment frame structures was developed [3], and the progressive collapse potential of steel moment frames was investigated considering catenary action of beams [4]. Kim and Park [5] developed a design procedure based on energy-balance equation to prevent progressive collapse of steel moment frames. Lee et al. [6] developed an energy-based analysis procedure for progressive collapse of moment frames considering catenary action of beams.

Most studies on progressive collapse of structures presented above have been conducted based on deterministic approaches where the nominal or average values of the design parameters were used. This study developed a reliability-based design procedure for steel moment frames using an energy-balance equation in such a way that a target reliability index for progressive collapse is satisfied in case a column is suddenly removed. The uncertainty of material properties was taken into account by considering the variation of design parameters such as yield strength, live load, and elastic modulus.

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II. Estimation of Reliability Index

For a realistic simulation of progressive collapse, the analysis process needs to include uncertain characteristics of material properties. Nevertheless, most recent studies have been conducted based on deterministic approaches where the nominal or average values of the design parameters were used. An application of the theory of probability to the structural analysis is one of the ways to deal with uncertain material properties which are considered as random variables [7]. The progressive collapse mechanism and the capacity of structures are affected by probabilistic properties of the design parameters and load combinations. The target reliability indices for design were determined using the FOSM (First Order Second Moment) method based on the assumption that the probability distributions of the design variables have the form of normal distribution. Table 1 shows the annual probability of occurrence of three-types of abnormal loads acting on building structures, such as gas explosions, bomb explosions, and automobile [8].

TABLE I
ANNUAL PROBABILITY OF OCCURRENCE OF ABNORMAL LOADS

Abnormal loads (H)	Probability (P(H))
Gas explosion	2×10^{-5}
Bomb explosion	2×10^{-6}
Automobile collision	6×10^{-4}

The CIRIA report [9] suggested the following function for the probability of structural collapse as a function of the social criterion factor, design life, and the mean number of people around a structure. Table 2 presents the social criterion factors for various natures of structures.

TABLE II
SOCIAL CRITERION FACTORS

Nature of structure	ς_s
Places of public assembly, dams	0.005
Domestic, office, trade and industry	0.05
Bridges	0.5
Tower, masts, offshore structures	5

III. Variation of Design Parameters and Output Responses

In this study the First-Order Second Moment (FOSM) method was applied to obtain the probabilistic distribution of vertical displacement response. In the FOSM method, means and standard deviations of input random variables (i.e. design

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parameters) are assumed and the mean and standard deviations of structural responses are obtained. The advantage of the FOSM method is that the analysis procedure is simpler than rigorous probabilistic methods such as the first-order reliability method, stochastic finite element method, and the Monte Carlo simulation method, while major probabilistic properties of the structural responses can be obtained accurately enough.. Table 3 shows the statistical data for some selected design variables, such as yield strengths of beams and columns, live load, and elastic modulus [10].

TABLE III
STATISTICAL PROPERTY OF DESIGN VARIABLES. (kN/cm^2)

Random variables	Mean	SD	COV (%)
Yield strength (beam)	23.50	1.24	5.30
Yield strength (column)	32.50	3.28	10.10
Live load	2.74	0.49	17.80
Elastic modulus	20594.00	679.60	3.30

IV. ENERGY-BASED DESIGN PROCEDURE

Progressive collapse occurs due to formation of collapse mechanism in all girders located in the bays from which a column is removed. In this paper the limit state for plastic rotation of girders given in the guideline shown in Table 4 is used to define progressive collapse. The external work is done by the gravity load imposed on the removed column and the vertical deflection. The internal work is computed by the plastic moments of the beams, which are initially unknown, multiplied by the rotation of the beams. The unknown beam plastic moment required to stabilize the structure can be obtained by equating the internal and the external works.

Figure 1 shows the deformed configuration of a framed structure with an internal column removed. The loss of a column results in the vertical deflection and the beam rotation. After the column is removed progressive collapse will not occur if the external work done by the gravity load is in equilibrium with the internal work done by the plastic rotation of beams at the vertical deflection less than the limit state. Eq. (1) represents the equilibrium of the external and internal works, from which the unknown plastic moment demand of the ith beam M_{pi} can be obtained using the vertical load P acting on the removed column and the virtual vertical deflection and the beam rotation [11]:

$$P \cdot \delta = \sum_{i=1}^{N} M_{pi} \cdot \theta_i \qquad (1)$$

where N is the number of plastic hinges. In case all spans have the same length and the same size of beams are used in all stories, the deflection can be expressed in terms of the rotation multiplied by the span length l, and the required beam plastic moment M_p is obtained as

$$M_p = \frac{Pl}{N} \qquad (2)$$

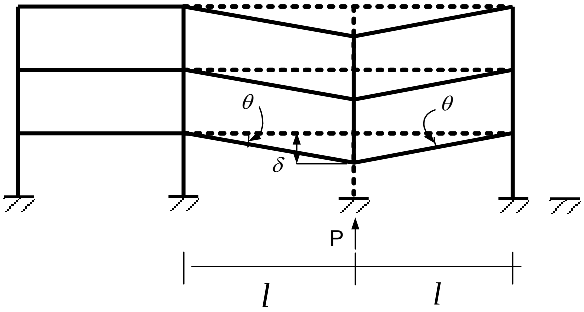


Fig. 1 Beam rotation and joint deflection caused by loss of a column

V. DESIGN OF MODEL STRUCTURES

The prototype structures for analysis models are the three-, nine-, and fifteen-story steel moment resisting frames with 3 bays and 3m story height. The structural plan of the prototype structure is shown in Fig. 2. The model structure was analyzed using the nonlinear analysis program code OpenSees [12]. Three different span lengths were considered in the analysis. The structures were designed with dead and live loads of $5.0kN/m^2$ and $3.0kN/m^2$, respectively, and seismic load with SDs and SD1 equal to 0.36g and 0.15g, respectively, with the response modification factor of 5.0. The beams and columns were made of SS400 ($Fy=23.5kN/m^2$) and SM490 ($Fy=32.4kN/cm^2$) steel, respectively. Both material and geometric nonlinearities were considered in the nonlinear dynamic analyses of the model structures. The beams and columns were modeled using the nonlinear beam-column elements with five integration points and 2% of post-yield stiffness. The catenary action of a beam caused by large deflection was considered using the 'Corotational' option in the OpenSees. For initiation of progressive collapse, one of the first-story columns was suddenly removed as shown in Fig. 3.

To simulate the performance of a structure with a column suddenly removed, the load combination shown in Fig. 3, dead load + 0.25×live load, was applied following the GSA guidelines. For this imposed load, the axial force, bending moment, and shear force acting on the column to be removed were computed first. Then the column was replaced by the point forces equivalent of its member forces. The forces were increased linearly for five seconds until they reached their full amounts, and were suddenly removed at seven seconds while the imposed load remained unchanged.

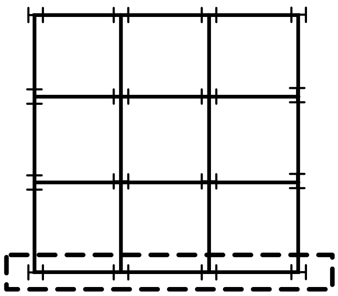


Fig. 2 Plan shape of model structure

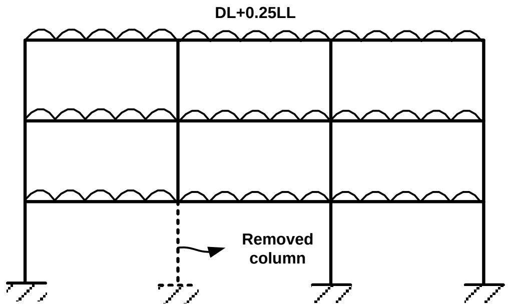


Fig. 3 Imposed load for dynamic analysis

VI. Computation of Modification Factors

Figure 4 shows the reliability indices of three-story structures with various span lengths as a function of modification factors. As the modification factor applied to the energy balance equation increases, the required plastic moment decreases. The reliability index was computed by nonlinear dynamic analysis of the model structures designed using the plastic moment computed in the energy balance equation. The horizontal dotted lines represent the target reliability indices for gas and bomb explosions.

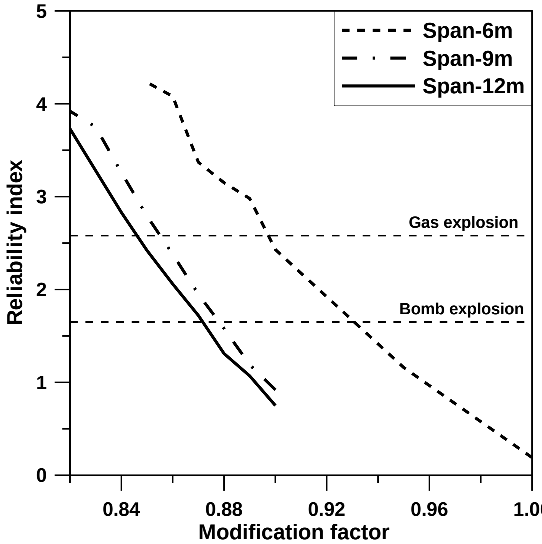


Fig. 4 Variation of a modification factor depending on a target reliability index

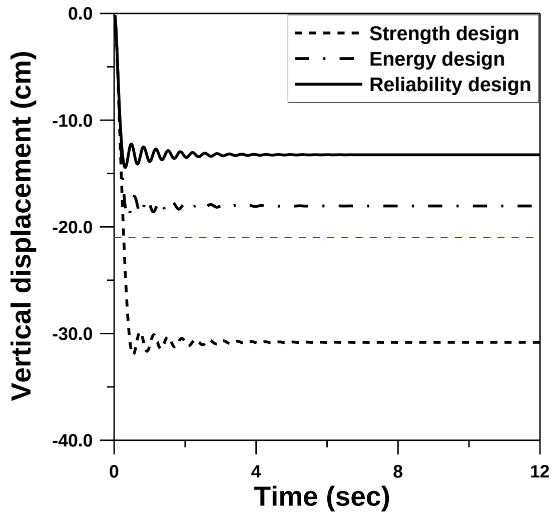
TABLE IV
MODIFICATION FACTOR FOR MODEL STRUCTURES WITH VARIOUS SPAN LENGTH

Abnormal loads	Span lengths		
	6m	9m	12m
Gas explosion	0.897	0.855	0.846
Bomb explosion	0.931	0.878	0.872

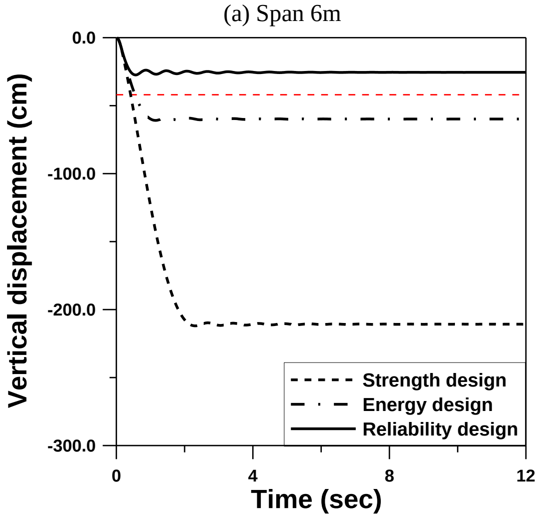
Table 4 shows the modification factors corresponding to the target reliability indices for abnormal loads obtained from Fig. 8. It can be observed that as the span length increases from six to twelve meters the reliability index decreases from 0.897 to 0.846 when the abnormal load considered is gas explosion. The reliability index decreases from 0.931 to 0.872 in case of bomb explosion. As the span length increases the capacity for resisting progressive collapse also decreases and so does the modification factor.

VII. Time Histories of Vertical Displacements

Figure 5 shows the time histories of the vertical displacements at the interior beam-column joints from which a column was suddenly removed. The displacements of the structures designed using the conventional strength-based design, the energy-based design, and the reliability-based design were compared. The limit states recommended in the GSA guidelines were also plotted as horizontal dotted lines. The vertical displacement responses were normalized by the limit states. In case of the three-story structure with 6m span, shown in Fig. 5(a), the maximum displacement of the structure designed by the conventional method exceeded the limit state, whereas those of the structures designed by the energy and reliability-based methods were less than the limit state. In the three-story structure with 12m span length, the conventionally designed structure became unstable as a result of column removal and the structure designed by the energy balance equation remained stable but the maximum displacement exceeded the limit state (Fig. 5(b)). In all cases the maximum displacements of the structures designed using the reliability-based method turned out to be about 70% of the limit state regardless of the number of story and the span length of the model structure.



(a) Span 6m



(b) Span 12m

Fig. 5 Time history of vertical deflection (3-story structures)

Table 5 shows the means and standard deviations of the vertical displacements of the three-story structures with various span lengths designed by the reliability-based energy method for gas explosion. It can be observed that as the span length increases the mean and standard deviation of the response also increases. The sum of the mean and the twice of the standard deviation resulted in displacement slightly smaller than the given limit state...

TABLE V
VERTICAL DISPLACEMENT OF THE 3-STORY STRUCTURE DESIGNED USING THE RELIABILITY-BASED PROCEDURE (UNIT: CM)

Span	6m	9m	12m
Mean (μ)	14.43	20.63	27.43
Standard deviation (σ)	2.60	4.11	5.29
$\mu + 2\sigma$	19.63	28.85	38.01
Limit state	21.00	31.50	42.00

VIII. CONCLUSION

According to the nonlinear time-history analysis results the maximum deflections of the model structures designed with conventional strength-based or energy-based design procedures did not always satisfied the given performance limit states, whereas those of the structures designed with the proposed method always satisfied the given limit states. In case of designing the three-story structure against progressive collapse caused by gas explosion, the modification factor applied to the energy-balance equation to satisfy the target reliability index reduced from 0.897 to 0.846 as the span length increased from 6m to 12m. It was also observed that the mean displacements plus twice the standard deviations of the maximum vertical displacements of the model structures designed using the proposed procedure were slightly less than the limit states.

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