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Sensitivity analysis of steel buildings subjected to column loss

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ABSTRACT

In this study, the sensitivity of design parameters of steel buildings subjected to progressive collapse is studied. To this end, design parameters such as yield strengths of beams, columns, and braces, live load, elastic modulus, and damping ratio were considered as random variables. The Monte Carlo simulation, the Tornado Diagram analysis, and the First-Order Second Moment method were applied to deal with the uncertainties involved in the design parameters. The analysis results showed that among the design variables beam yield strength was ultimately the most important design parameter in the moment-resisting frame buildings while the column yield strength was the most important design parameter in the dual system building. Sensitivity of the vertical displacement to uncertain member strength showed that progressive collapse mechanisms of the moment-resisting frame buildings and the dual system building completely differed due to different patterns of the vertical load redistribution.

1. Introduction

When a structure is subjected to unexpected loads such as explosion, impact, fire, etc. that are not considered in the normal design process, the structure may become vulnerable. The phenomenon whereby the failure of one or more load-resisting structural members due to an unexpected load leads to the collapse of the entire structure, especially in a domino-like way, is commonly called progressive collapse [1].

The collapse of the Alfred P. Murrah Building in 1995 and the World Trade Center (WTC) Tower in 2001 are examples of progressive collapse due to a car-bombing and an aircraft impact, respectively. Before the collapse of the WTC, research on progressive collapse had only been conducted by a limited number of researchers because the probability that such an abnormal loading event would occur and that it would trigger progressive collapse was very low. However, the collapse of the WTC, where more than 2000 civilians lost their lives, reminded structural engineers that the mechanism of progressive collapse needs to be thoroughly understood to prevent such a disaster recurring in the future.

To prevent the progressive collapse caused by abnormal loads, the National Building Code of Canada [2] specified requirements for the design of major elements, the establishment of connection elements, and ways of providing load transfer paths. Eurocode 1 [3] presented a design standard for selecting plan types for preventing progressive collapse and recommended that buildings should be integrated. In the United States, specific provisions related to progressive collapse have not yet been provided in design codes such as the International Building Code [4]. However, the American Concrete Institute [5] requires structural integrity (for example, continuity insurance of reinforcing bars) so that partial damage by abnormal loads does not result in the collapse of the entire structure. The ASCE 7-05 [6] also recommends a design method, a load combination, and structural integrity, as does ACI 318. The General Service Administration (GSA) presented a practical guideline for design to reduce the collapse potential of federal buildings [7]. The Department of Defense (DoD) also presented a guideline for new and existing DoD buildings [8]. These guidelines address design procedures and analysis methodology for progressive collapse.

Research on progressive collapse can be categorized according to two different approaches: (1) developing structural systems that prevent progressive collapse, and (2) developing an analysis methodology. Crawford [9] proposed the use of connection details such as Side Plate™, developed for earthquakes, the use of cables imbedded in reinforced concrete beams to activate catenary action, and the use of mega-trusses in high-rise buildings to resist progressive collapse. Suzuki et al. [10] showed that

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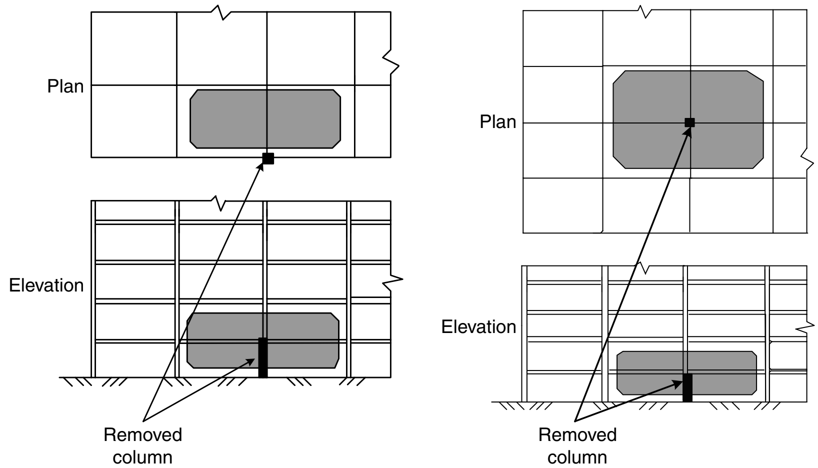


(a) Exterior consideration.

(b) Interior consideration.

Fig. 1. Allowable collapse zone [7].

the use of hat-bracing at the top of structures may increase the resistance to progressive collapse. Hayes Jr. et al. [11] investigated the relationship between seismic design and the blast or progressive collapse-resisting capacity. They mentioned that the seismic design details developed for special moment frames in high seismic zones would provide better resistance to external explosion or impact load than the less-rigorous design details of ordinary moment frames. Khandelwal et al. [12] also investigated the mechanism of the progressive collapse of seismically designed braced steel frames.

Both linear and nonlinear analysis methods can be used to simulate progressive collapse. The linear analysis method can be readily adopted to the alternative path method [7] where the demand-capacity ratio of the structure is evaluated repeatedly. However, Powell [13] proposed that the nonlinear analysis method should be used for progressive collapse because the result of the linear analysis can be too conservative and is sensitive to input parameters.

For realistic simulation of structural performance, the analysis process needs to include uncertain characteristics of material properties. Nevertheless, most recent research on progressive collapse of structures has been conducted based on deterministic approaches where the nominal or average values of the design parameters were used [14]. 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 [15]. The effect of variability of uncertain design parameters on structural behaviors can be estimated by a sensitivity analysis. Sensitivity analysis has been used for earthquake engineering to estimate sensitive design parameters to the seismic response of buildings [15]. Recently Park and Kim [16] carried out fragility analysis of steel structures subjected to progressive collapse considering the probability distribution of material properties. The progressive collapse mechanism and the capacity of structures can be affected by the probabilistic properties of the design parameters and load combinations. The sensitivity analysis is necessary to understand which design parameters are more important to progressive collapse than others.

The objective of this study is to determine the important design parameters and structural members for the progressive collapse mechanism of buildings. To this end, three different probabilistic approaches were used on steel moment frame buildings and dual system buildings of various stories. Uncertainties associated with material properties and member capacities were considered in order to determine the influential material properties and members for the progressive collapse of the analysis model buildings.

2. Analysis methodology

2.1. Analysis method for progressive collapse

Progressive collapse refers to the phenomenon whereby local damage of structural elements caused by abnormal loads results in global collapse of the structure. To carry out analysis for progressive collapse, it is necessary to track progressive member failures. In the design guidelines of the GSA and the DoD, an analysis method for progressive collapse, called the alternative path method, is presented. In this method, damaged structural members are removed from the model structure and the loads originally supported by the lost members are redistributed to neighboring members. In this way, progressive member failures can be evaluated. However, this procedure requires repetitive analysis until progressive member failure stops and the structure is stabilized.

The guidelines define the allowable collapse zone, as shown in Fig. 1, such that if a member fails outside of this zone, the probability of progressive collapse is considered to be high. In this study, the onset of progressive collapse was determined by the failure of a structural member located outside of the allowable collapse zone when a column located in the zone was removed.

For numerical simulations of the progressive collapse, nonlinear static and dynamic analyses were conducted [17,18]. An incremental vertical displacement was applied downward of the node, where the damaged column was removed and the corresponding applied vertical load was recorded. The gravity load is a combination of dead load (DL) and live load (LL). The load combination was assumed to be DL + 0.25LL, as shown in Fig. 2. For dynamic analysis, all member forces were first obtained from the full structural model subjected to the applied load (DL + 0.25LL). The structure was then re-modeled with a column removed and its member forces applied to the structure as a lamp force to maintain equilibrium (Fig. 3). The forces were suddenly removed after 7 seconds to initiate progressive collapse as shown in Fig. 3, where W denotes member forces of the lost column. In this way, the progressive collapse analysis started from the moment the structure was already deformed by the applied load.

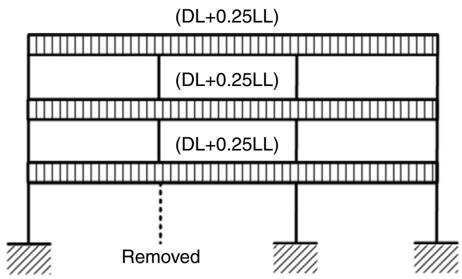


Fig. 2. Load combination for structural analyses.

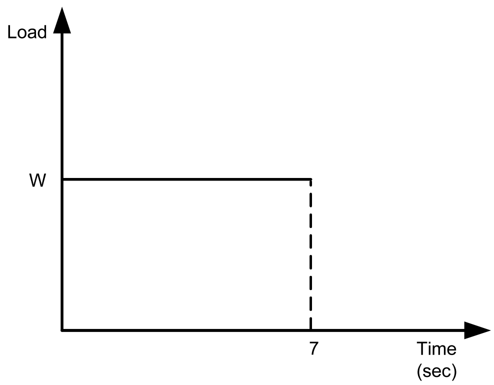


Fig. 3. Applied load for dynamic progressive collapse analysis.

2.2. Sensitivity analysis method

For sensitivity analysis of structures associated with progressive collapse, the variability of structural response due to the variability of structural properties was evaluated. To this end, three different methods based on the probability theory were adopted: (1) Monte Carlo simulation (MCS), (2) Tornado Diagram Analysis (TDA), and (3) the First-Order Second Moment (FOSM) method.

MCS is one of the methods widely used to analyze random problems. In this method, random variables are represented by sets of deterministic values that are used to produce sets of deterministic outputs. Then probabilistic forms of outputs are constructed. Because of its robustness, MCS is frequently used to validate other probabilistic analysis methods [19].

TDA is one of the sensitivity analysis tools commonly used in decision analysis. Porter et al. [20] applied it to the seismic sensitivity analysis of structures. In TDA, the upper and lower bounds of a random variable are selected and the corresponding structural responses are obtained. The difference between such structural responses, referred to as swing, is considered as a measure of sensitivity. The schematic view of this process is depicted in Fig. 4.

In the FOSM method, means and standard deviations (SD) of random variables are assumed and the mean and SD of structural responses are obtained where SD can be used as a measure of sensitivity [21]. 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 MCS method, while major probabilistic properties of the structural responses can be obtained. The detailed analysis procedure of the FOSM method can be found elsewhere [21].

3. Case study building: three-story building

3.1. Descriptions of analysis model

The selected case study building is a three-story, three-bay, steel moment-resisting frame building designed according to the Korean Building Code (KBC) 2005 [22]. The structure was designed

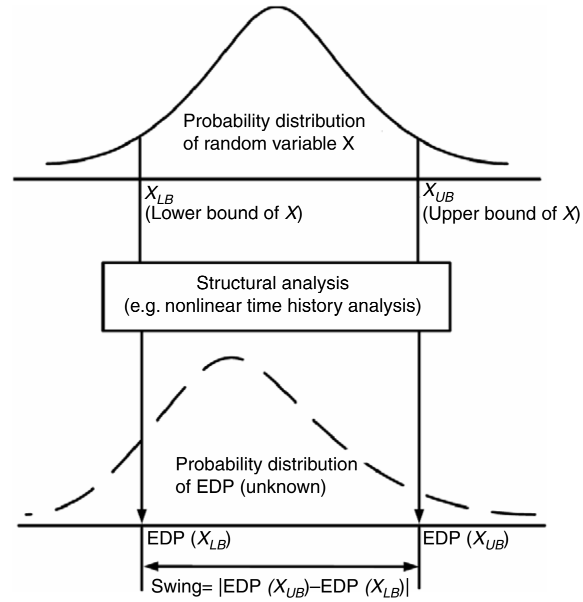


Fig. 4. Tornado diagram analysis procedure [17].

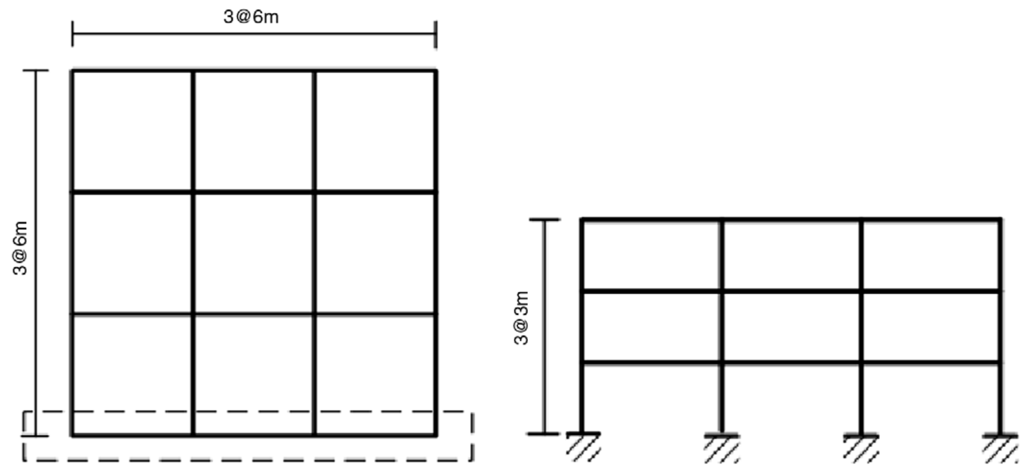
with the dead load of 5 kN/m^2 and the live load of 2.5 kN/m^2. Interior columns are composed of H300 $\times$ 300 $\times$ 10 $\times$ 16, exterior columns are of H250 $\times$ 250 $\times$ 14 $\times$ 14, and beams are of H300 $\times$ 120 $\times$ 8 $\times$ 13. SM490 (yield strength $=$ 32.4 kN/cm^2) and SS400 steel (yield strength $=$ 23.5 kN/cm^2) were used for columns and beams, respectively. Fig. 5 shows the plan and elevation views of the case study building.

For the purpose of analysis, it was assumed that an exterior column of the building failed due to an abnormal load. One of the exterior frames (Fig. 5(a)) of the building was analyzed by OpenSees, a structural analysis software framework specialized for nonlinear analyses [23]. Beams and columns were modeled using the 'nonlinearBeamColumn' element from the OpenSees element library where five Gauss points were used for numerical integration of the element. The beam–column connection was modeled so that the stiffness of the panel zone could be considered [24]. The material model used for the structure was 'Steel01' in the OpenSees material library, of which the bilinear stress–strain relation with 2% strain hardening ratio is depicted in Fig. 6. To simulate catenary action due to large deformations, the 'Corotational' option was used for the element transformation. Two cases of initial conditions that may trigger the progressive collapse were considered in this study, as shown in Fig. 7, where (1) an interior column was damaged (Fig. 7(a)) or (2) an exterior column was damaged (Fig. 7(b)).

3.2. Sensitivity analysis

Sensitivity of the vertical displacement to uncertain design parameters was studied in three different approaches. Uncertain design parameters considered in this study were yield strengths of the beam and the column, live load, damping ratio, and elastic modulus of the steel. Statistical properties of these random variables were assumed based on the available literature and are summarized in Table 1 [15,25,26]. Among the five random variables, the damping ratio was considered only in the dynamic analysis. In the TDA and FOSM methods, the SD of the vertical displacement due to the SD of a random variable was selected as a measure of sensitivity to the given variable. In the process of computing the sensitivity of a certain random variable, the other random variables were fixed at their mean values.

In the MCS analysis, the sample size must be larger than a certain value, which is called the minimum sample size, to



(a) Plan. (b) Elevation.

Fig. 5. Plan and elevation of the three-story case study building.

Table 1
Statistical property of random variables.

Random variables	Mean	SD	COV (%)	Dist'n type
Yield strength (beam and brace)	23.5 kN/cm^2	1.24 kN/cm^2	5.3	Lognormal
Yield strength (column)	32.5 kN/cm^2	3.28 kN/cm^2	10.1	Lognormal
Live load	2.74 kN/m^2	0.488 kN/m^2	17.8	Lognormal
Damping ratio	5%	2%	40.0	Lognormal
Elastic modulus	20 594.0 kN/cm^2	679.602 kN/cm^2	3.3	Normal

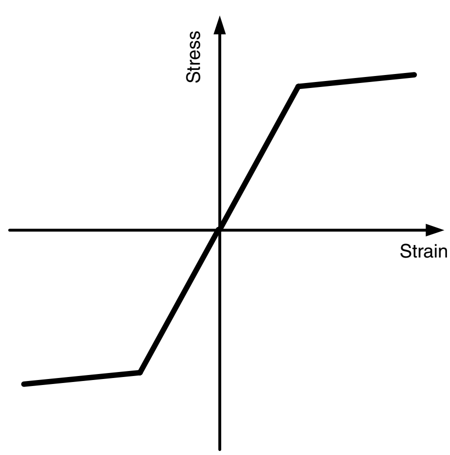
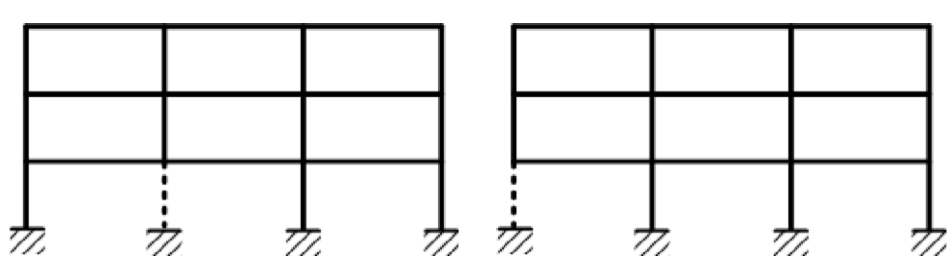


Fig. 6. Stress–strain relationship of the Steel01 material model of OpenSees.

guarantee a certain level of reliability. For this, a convergence test was performed to determine the minimum sample size in such a way that the selected tolerance, namely the coefficient of variability (COV) of 5%, was satisfied. The minimum sample size for each of the static and dynamic analyses was determined with respect to the random variable with the largest COV. Therefore, the live load for the static and the damping ratio for dynamic analyses were considered for the convergence test. Figs. 8 and 9 show the result of convergence tests of means and SDs of the vertical displacement where the minimum sample sizes for static and dynamic analyses were observed as 765 and 3060, respectively. Accordingly, the sample sizes of 1000 and 3100 were selected for the static and dynamic analyses, respectively.

Variability of the vertical displacement due to the random variables is presented in the form of a tornado diagram. As shown in Fig. 4, variability of the vertical displacement due to variability of a random variable is defined as 'swing'. In the tornado diagram, swings due to various random variables are displayed in the descending order of the swing size from top to bottom. A larger swing size implies a larger effect of the corresponding random variable on the vertical displacement. To compare sensitivity according to the three different methods, results from the MCS and FOSM methods were also presented in the same tornado diagrams as shown in Figs. 10 and 11. In these diagrams, tornado diagrams were developed based on the mean $\pm$ 2SD of the random variables.

For both static and dynamic analyses, the vertical displacement was ultimately the most sensitive to the yield strength of the beam regardless of the location of the initially damaged column. The live



(a) Loss of interior column. (b) Loss of exterior column.

Fig. 7. Two cases of initial conditions for collapse analysis.

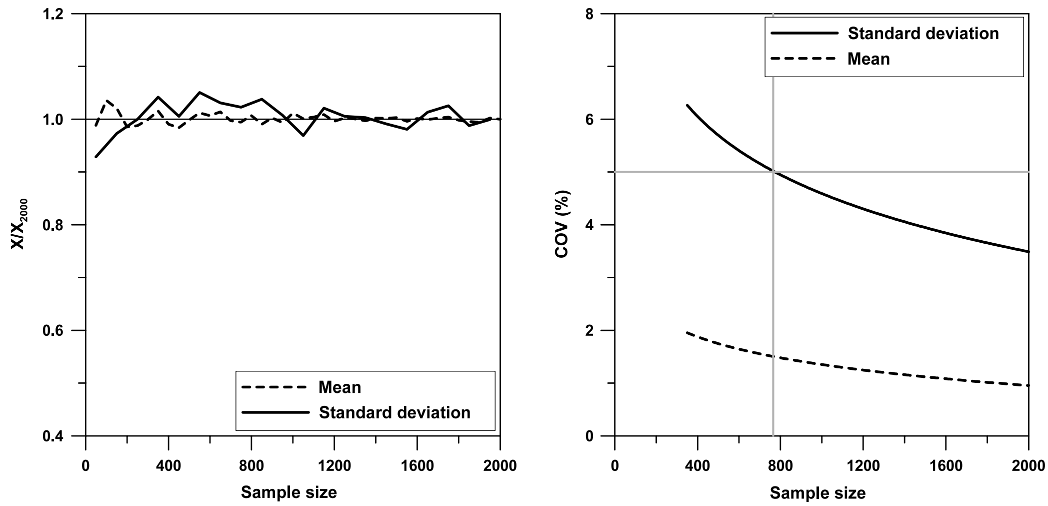


Fig. 8. Convergence test of MCS for static analysis.

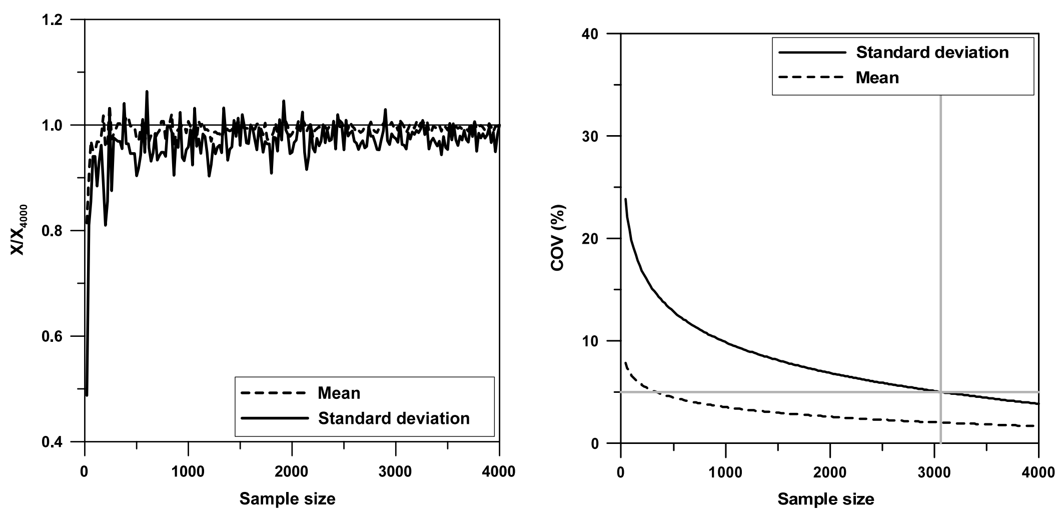
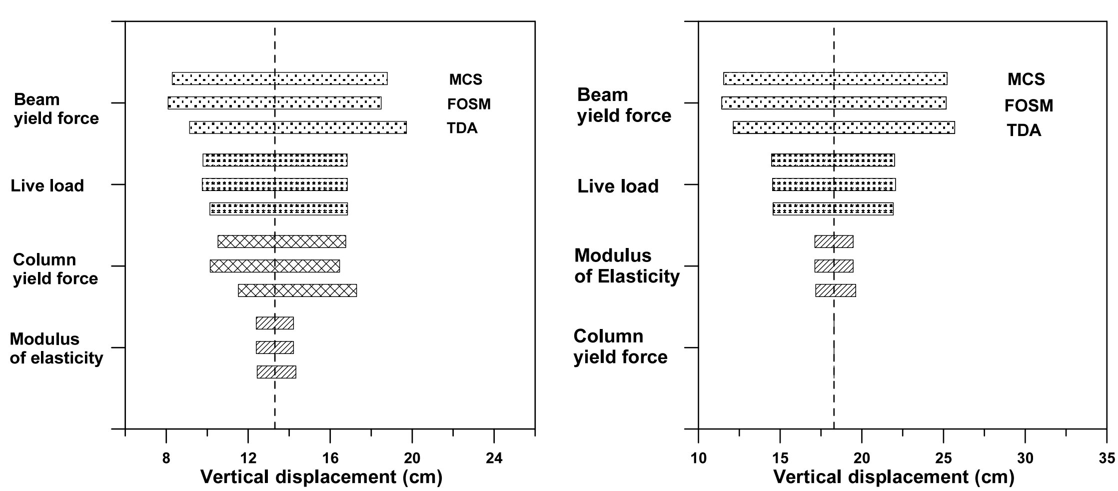


Fig. 9. Convergence test of MCS for dynamic analysis.



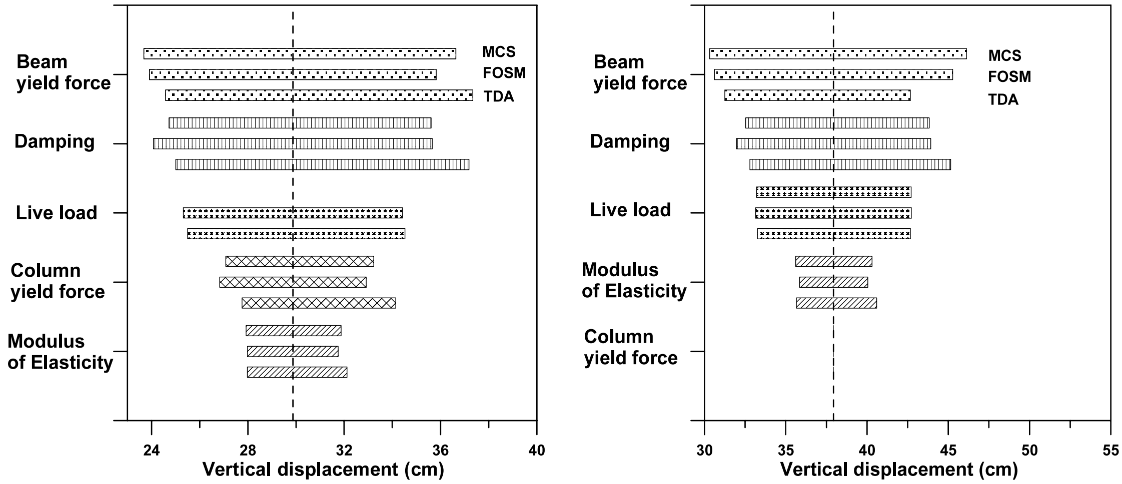
(a) Removal of interior column.

(b) Removal of exterior column.

Fig. 10. Vertical displacement sensitivity of the three-story building: static analysis.

load was the second most sensitive random variable to the vertical displacement in the static analysis, while the damping ratio was the second most sensitive random variable in the dynamic analysis.

The accuracy of the TDA and the FOSM method with respect to the result of the MCS analysis was then investigated. Table 2 summarizes the errors of swing sizes for the TDA and the FOSM



(a) Removal of interior column.

(b) Removal of exterior column.

Fig. 11. Vertical displacement sensitivity of the three-story building: dynamic analysis.

Table 2
Errors (%) of swing sizes for the TDA and FOSM methods.

Removal of interior column (static analysis)		
	TDA	FOSM
Yield strength (beam)	−0.76	0.95
Yield strength (column)	7.53	−1.28
Live load	4.55	−0.57
Elastic modulus	−3.30	0.00
Removal of interior column (dynamic analysis)		
Yield strength (beam)	1.65	8.91
Yield strength (column)	−3.74	1.14
Live load	2.06	1.19
Elastic modulus	−5.08	4.57
Damping ratio	−11.75	−6.34
Removal of exterior column (static analysis)		
Yield strength (beam)	0.88	−0.51
Yield strength (column)	0.00	0.00
Live load	2.26	0.13
Elastic modulus	−3.83	0.43
Removal of exterior column (dynamic analysis)		
Yield strength (beam)	38.35	7.75
Yield strength (column)	0.00	0.00
Live load	1.16	−0.63
Elastic modulus	−4.89	10.64
Damping ratio	−9.38	−5.84

method. The largest error of the TDA result, 38.35%, occurred with the yield strength random variable in the dynamic analysis for the case where the exterior column is removed, while the largest error of the FOSM method, 10.64%, occurred with the elastic modulus random variable in the dynamic analysis for the case where the exterior column is removed, too. The error in the latter case was, however, not significant because, of the five random variables, the corresponding random variable was the fourth most important. Therefore the FOSM method can be used for sensitivity analysis of progressive collapse with almost the same level of accuracy as the MCS analysis.

4. Case study building: ten-story buildings

4.1. Building description and modeling

While the MCS method, which generally requires considerable computational effort, may be affordable for the progressive collapse analysis of a three-story building, it may be impractical for use on a ten-story building. For this reason the FOSM method was used for the sensitivity analysis of the progressive collapse of the ten-story buildings. Two different structural systems, the moment-resisting frame system and the dual system with moment-resisting and braced frames, were considered in this study. Fig. 12 shows the plan and the elevation views of the ten-story case study buildings designed according to the KBC 2005 [21]. For each structural system, an interior frame designated by the dotted lines in Fig. 12 was selected for structural analysis. The mass of the building was assumed to be lumped at nodes and the exterior and interior nodal masses were assumed to be 0.077 kN s^2/cm and 0.154 kN s^2/cm, respectively. Design loads are summarized in Table 3. It should be noted that these buildings were designed for wind and earthquake loads as well as gravity load.

Table 3
Design loads for ten-story case study buildings.

Gravity load	
Dead load	3.73 kN/m^2
Live load	2.74 kN/m^2
Wind load	
Exposure category	B
Basic wind speed	30 m/s
Importance factor	1.0
Average roof height	360 m
Gust factor	2.2
Earthquake load	
Seismic zone	0.11
Site class	Sb
Seismic use group	1
Importance factor	1.2
S_{ds}	0.3657 g
S_{d1}	0.1463 g
R factor	6 (steel moment frame) 5 (dual system)
Fundamental period	1.249 s

The ten-story moment-resisting frame was modeled by the same modeling strategy as that of the three-story case study building. The braces in the dual system were modeled in such a way that the buckling mode of braces could be simulated. For this, braces were modeled using the 'nonlinearBeamColumn' element from the OpenSees element library with an initial imperfection in the middle of the bracing members as shown in Fig. 13. In this study, 1/1000 was assumed as the initial imperfection.

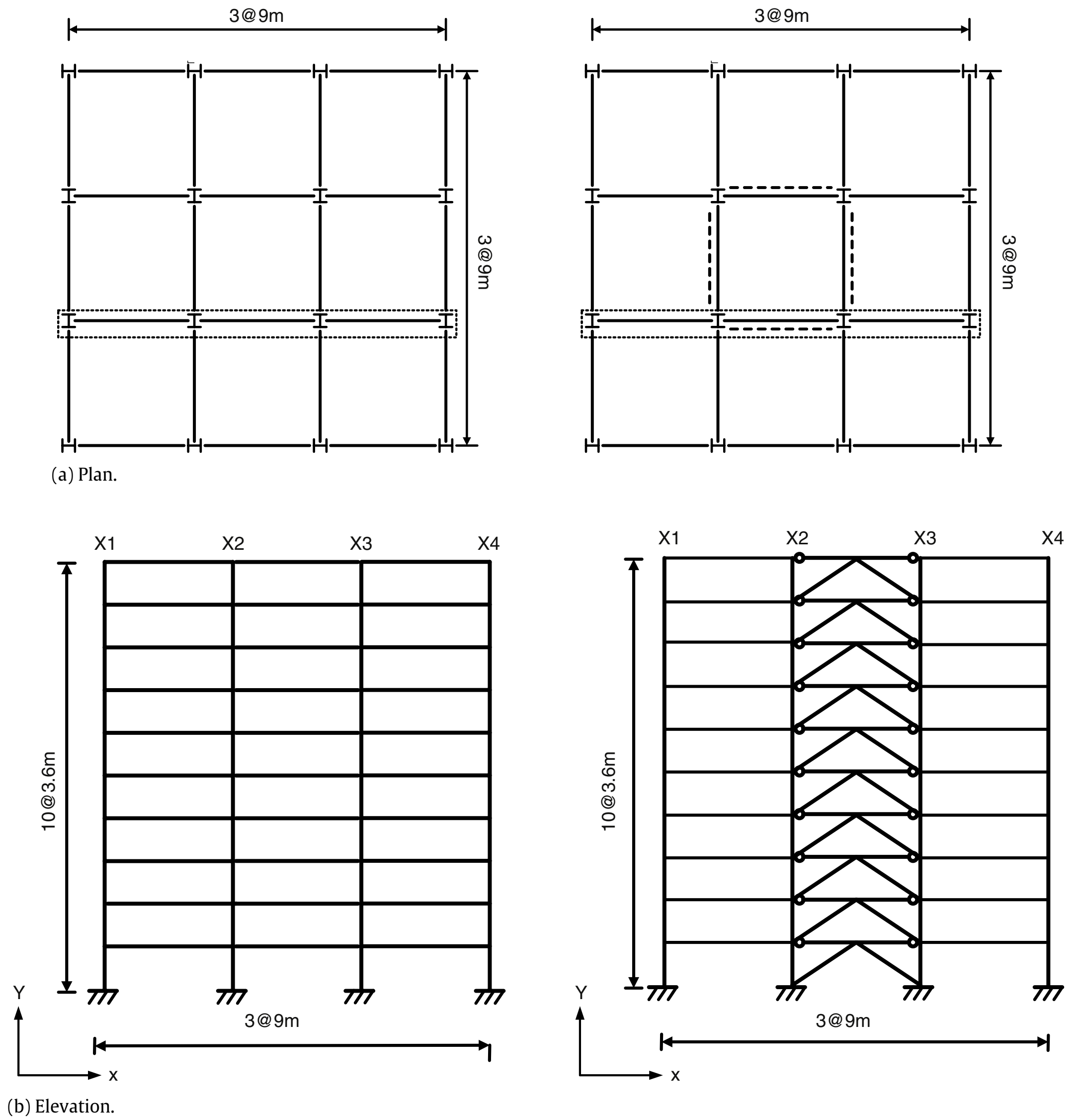


Fig. 12. Plan and elevation view of the ten-story case study buildings.

4.2. Sensitivity to uncertain parameters

Similar to the three-story building case, sensitivity of the vertical displacement to uncertain design parameters was investigated. The random variables used were identical to the three-story building case, with the exception of the yield strength of the bracing members. Fig. 14 shows the tornado diagram for sensitivity of the moment-resisting frame building from the static analysis. Similar to the result of the three-story building, the beam yield strength was observed as the most important design parameter while the live load was the second most important parameter. In the dynamic analysis, the damping ratio and the beam yield strength were the two most important design parameters as shown in Fig. 15.

For the case of the dual system, it is interesting to note that the variability of the brace yield strength did not affect variability of the vertical displacement for both static and dynamic analyses as shown in Figs. 16 and 17, respectively. In the middle bay, axial forces in bracing members are transferred (or redistributed) to neighboring columns after the loss of a first-story column. One of these columns may yield while bracing members are still in the elastic state. Similar to the case of the moment-resisting frame building, the damping ratio is the most important design parameter for the dual system building in dynamic analysis as shown in Fig. 17.

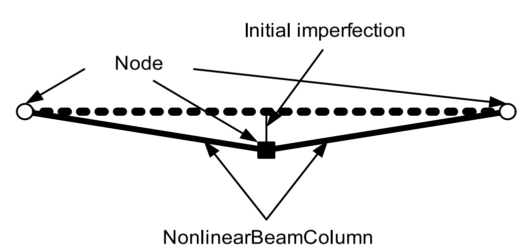
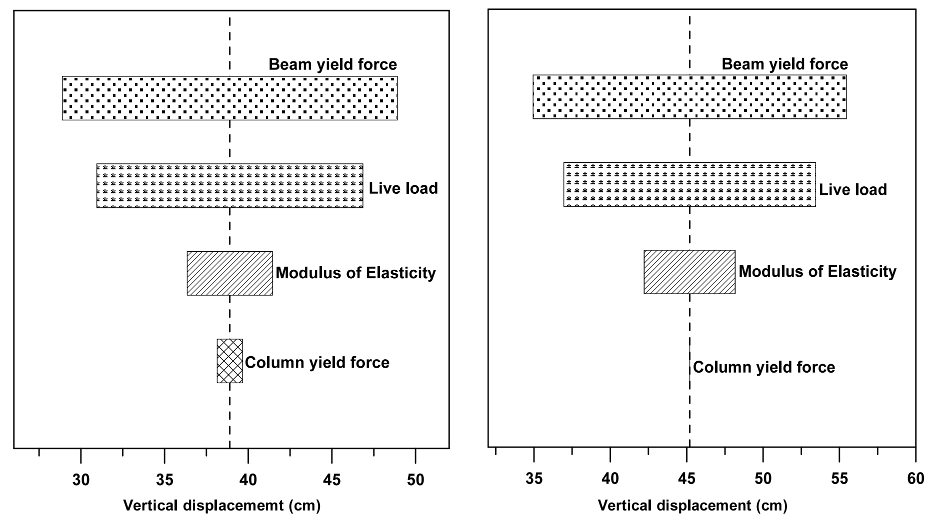


Fig. 13. Modeling of the bracing members.

4.3. Sensitivity to uncertain members

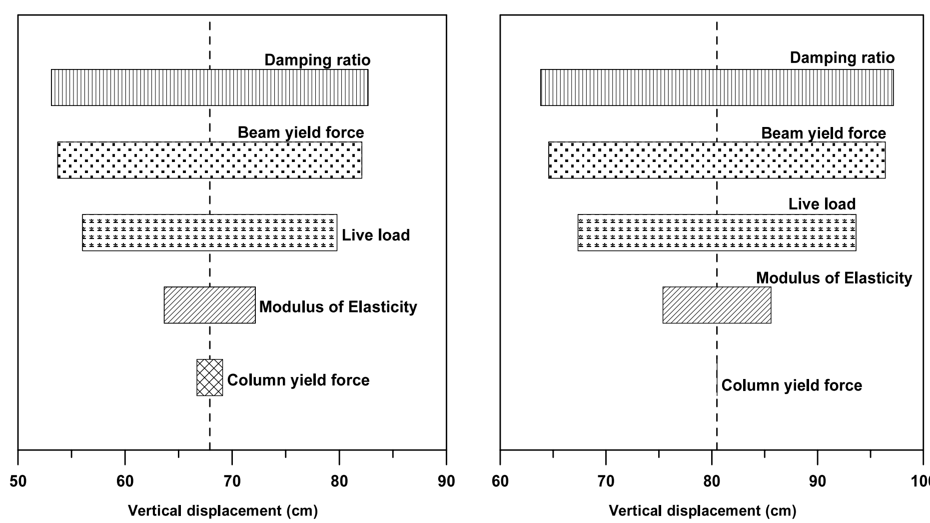
When a member in a building structure is significantly damaged, loads are redistributed to neighboring members, which may lead to the progressive collapse of the building. If member strengths are uncertain, the mechanism of the progressive collapse is also uncertain. In this section, the yield strength of each structural member was considered as a random variable to



(a) Removal of interior column.

(b) Removal of exterior column.

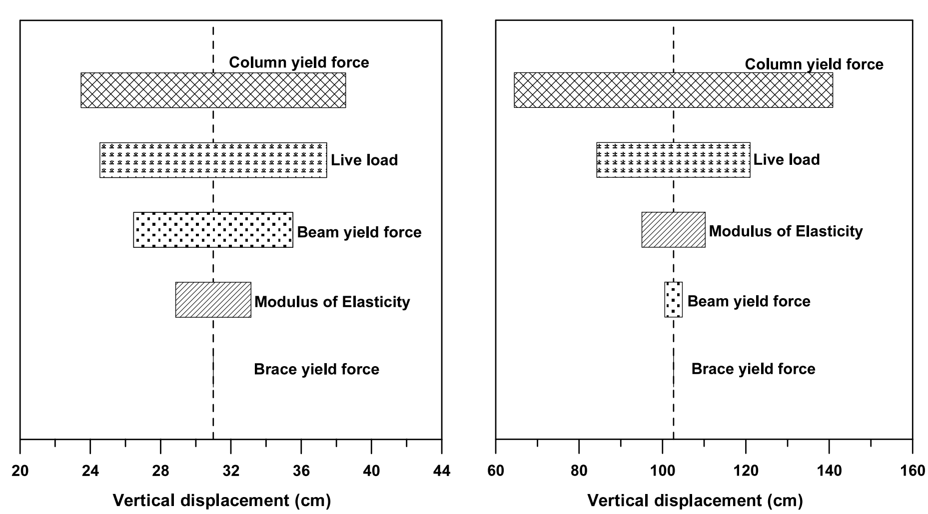
Fig. 14. Vertical displacement sensitivity of the ten-story moment-resisting frame building: static analysis.



(a) Removal of interior column.

(b) Removal of exterior column.

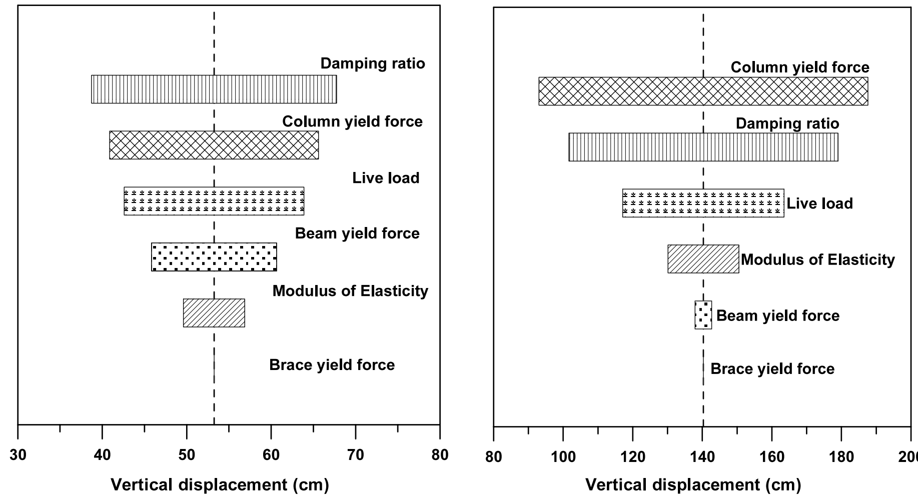
Fig. 15. Vertical displacement sensitivity of the ten-story moment-resisting frame building: dynamic analysis.



(a) Removal of interior column.

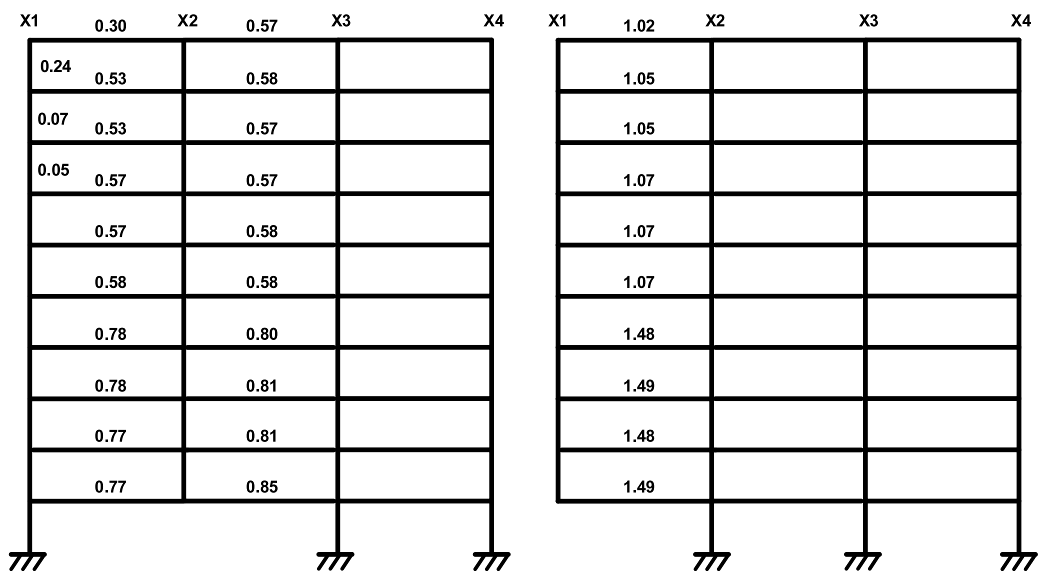
(b) Removal of exterior column.

Fig. 16. Vertical displacement sensitivity of the ten-story dual system building: static analysis.



(a) Removal of interior column. (b) Removal of exterior column.

Fig. 17. Vertical displacement sensitivity of the ten-story dual system building: dynamic analysis.



(a) Removal of interior column. (b) Removal of exterior column.

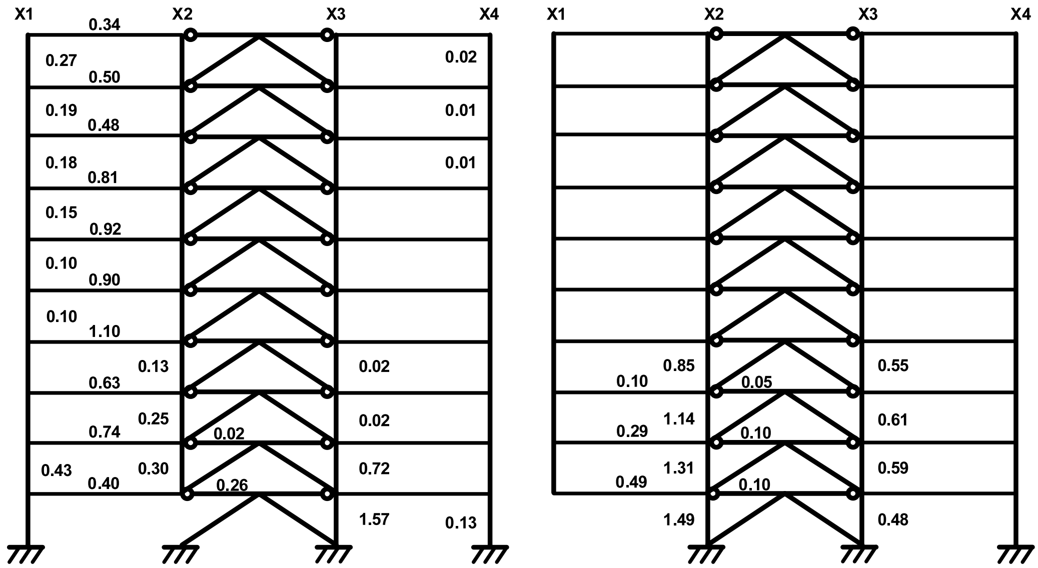
Fig. 18. Coefficient of variability of the moment-resisting frame to uncertain member strengths.

evaluate its effect on the sensitivity of the vertical displacement. For this, the FOSM method with the nonlinear static analysis procedure was used. It is noted that the live load and elastic modulus were assumed to be deterministic design parameters.

Fig. 18 shows the sensitivity of the vertical displacement of the moment-resisting frame building to the uncertain strength of members, where the numbers on beams denote the COV of the vertical displacement due to the strength variability of the corresponding members. When an interior column was removed, the beams located in the bays from which a column was removed were ultimately important to the variability of the vertical displacement (Fig. 18(a)). Similar to the case of interior column removal, the beams of the exterior bay containing the removed column were important when an exterior column was removed as shown in Fig. 18(b). For both column removal cases, the beams located in the lower stories were more influential to the variability of the vertical displacement than those of the upper stories. Column strengths were not very influential to the variability of the vertical displacement in the interior column removal case (Fig. 18(a)), while the vertical displacement was not sensitive at all to the variability of the column strength in the exterior column removal case (Fig. 18(b)).

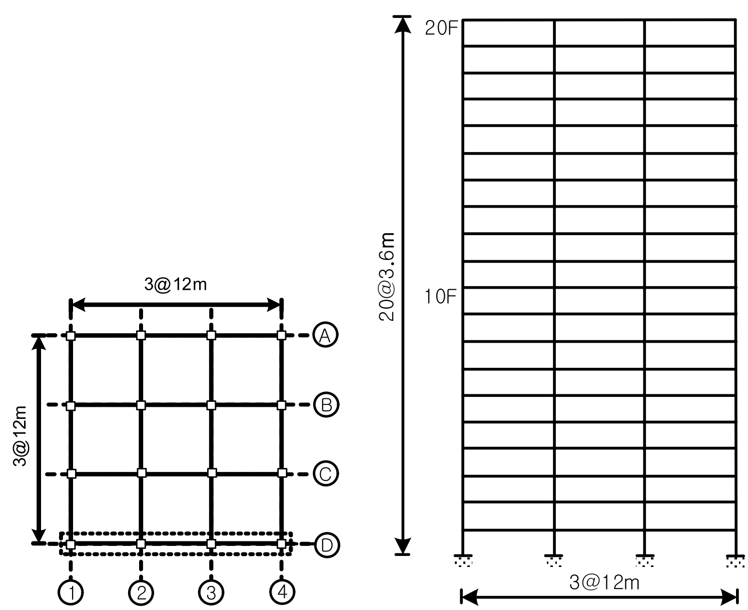
When an interior column was removed from the dual system building, the vertical displacement was most sensitive to the variability of the strength of the beams in the exterior bay as shown in Fig. 19(a). The beams in the first two floors in the middle bay were also influential to the sensitivity, and this is attributed to the load redistribution from the removed column. It can be observed that the influence of the columns in the X1 line of the dual system was larger than that of the moment-resisting frame. This is because, in the dual system, more loads were redistributed to the beams rather than to the braces and then to the exterior columns. On the other hand, the load was somewhat evenly redistributed to the neighboring beams on the left and the right side of the removed column in the moment-resisting frame. The columns located in the X2 and the X3 column lines in the first four stories were also influential because the load was redistributed to the braces and then to the neighboring columns.

When an exterior column was removed from the dual system building, some of the lower story beams and columns were



(a) Removal of interior column. (b) Removal of exterior column.

Fig. 19. Coefficient of variability of the dual system building to uncertain member strengths.



(a) Structural plan. (b) Elevation view.

Fig. 20. Twenty-story analysis model structure.

observed as influential members to the vertical displacement as shown in Fig. 19(b). It is interesting to note that the progressive collapse mechanisms of the moment-resisting and dual system buildings completely differed (Figs. 18 and 19) due to the different patterns of the load redistribution.

5. Case study building: twenty-story building

In this section, a FOSM analysis is conducted to obtain sensitivity of vertical displacement to the variability of yield strength of beams. Fig. 20 shows the plan and the elevation of the twenty-story case study building designed according to the KBC 2005 [21]. Nonlinear static pushdown analyses were carried out by removing a first-story exterior and an interior column one at a time to obtain the force–displacement relationship of the model structure. The mean and SD of the member yield strength used in the analysis are 23.5 kN/cm^2 and 1.245 kN/cm^2, respectively, and the mean $\pm$ 2SD of the vertical displacement was computed to represent the sensitivity of the response.

Fig. 21 depicts the pushdown curve of the model structure. The maximum load factor exceeding 1.0 implies that the imposed load specified in the GSA guideline can be supported by the structure after a column is suddenly removed. It can be observed that the removal of an interior column results in higher strength due mainly to higher redundancy of the structural members involved in resisting the progressive collapse. Fig. 22 shows the plastic hinge formation at beam ends when the rotation of the first-story beam reached 2.5% radian, which corresponds to the collapse prevention limit state. It can be observed that all beams located in the damaged bays yielded, and that the rotations of the beams located in the lowest two stories reached the limit state when the exterior or the

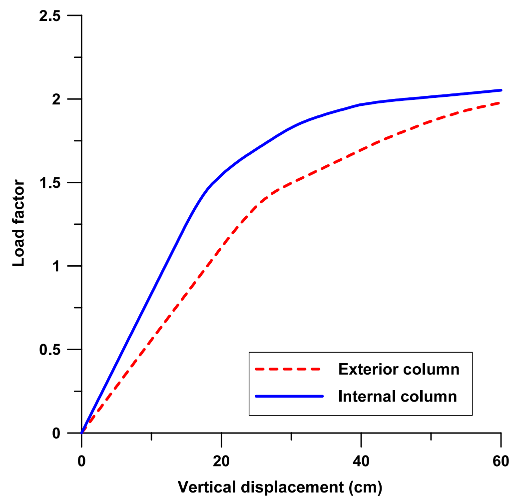


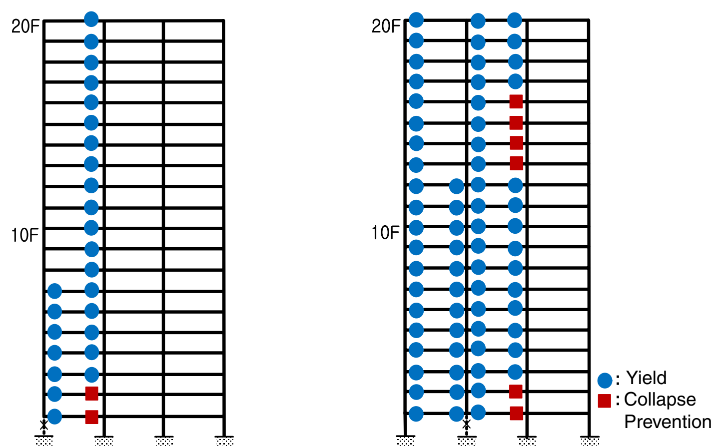
Fig. 21. Force–vertical displacement relationship when a first-story column is removed.

interior column was removed. The occurrence of large beam-end rotations observed in the beams above the 13th story (Fig. 22(b)) is because, from that story, the beam sizes are reduced.

Fig. 23 illustrates the sensitivity analysis results in terms of the tornado diagram when a first-story column was removed. The horizontal bars represent the variability of the vertical displacement (the mean $\pm$ 2SD) due to the variability of the yield strengths of the beams at the corresponding stories. When an exterior column was removed, the variability of the vertical displacement due to variability of the yield strength of the beams at exterior bays generally increased at lower stories. When an interior column was removed, the variability of the vertical displacement due to variability of the yield strength of the beams at an exterior bay were large at the lower thirteen stories and significantly decreased at higher stories. These results imply that the vertical displacement caused by column removal is more sensitive to the variability of the yield strength of the beams located at lower stories, and that the beams in lower stories play a more important role in resisting progressive collapse.

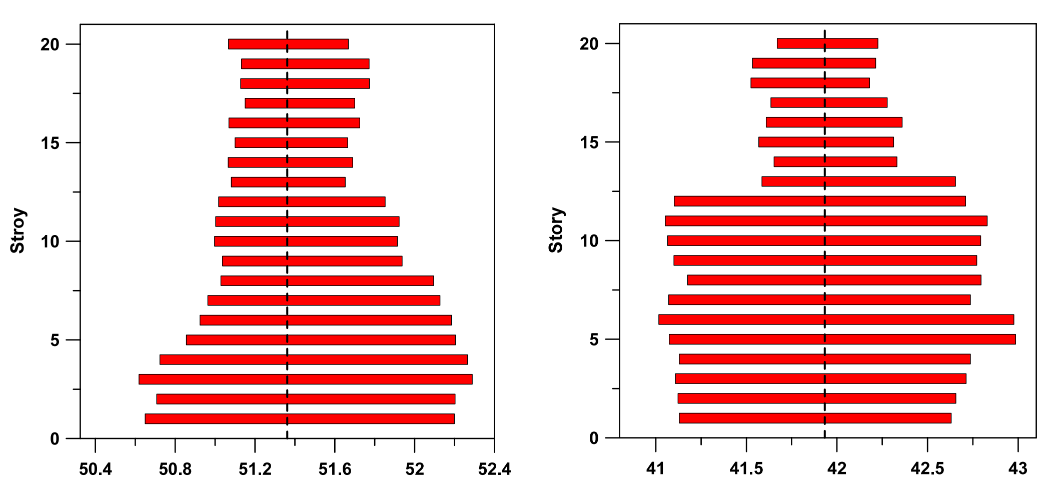
6. Conclusions

Sensitivity of the progressive collapse mechanism to uncertain design parameters of steel buildings was investigated using MCS, TDA, and the FOSM method. Yield strengths of structural members, live load, elastic modulus, and damping ratio were considered as random variables. One of the first-story columns was removed



(a) Removal of interior column. (b) Removal of exterior column.

Fig. 22. Plastic hinge rotation at beam ends.



(a) Removal of interior column. (b) Removal of exterior column.

Fig. 23. Variation of vertical displacement at exterior bay due to variation of beam yield strength at collapse prevention limit state.

to initiate progressive collapse. Two-dimensional nonlinear static and dynamic analyses were conducted within the procedure of the FOSM method to deal with uncertainties of design parameters and to evaluate the variability of structural responses in terms of the vertical displacement. This methodology was applied to a three-story, ten-story, and twenty-story moment-resisting frame buildings, and a ten-story dual system building. For sensitivity analysis of the three-story case study buildings, three different methods, the TDA method, the MCS method, and the FOSM method were applied to validate the efficiency of the FOSM method to the present examples. Sensitivity analyses of ten-story and twenty-story case study buildings were then conducted by the FOSM method.

The beam yield strength and damping ratio were observed as the most influential design parameter to the vertical displacement in the three-story moment frame while elastic modulus and column yield strength were not influential parameters. From static pushdown analysis of the ten-story buildings, the beam yield strength and the column yield strength were observed as the most influential design parameters for the moment-resisting frame and the dual system buildings, respectively. The damping ratio was the most important parameter in dynamic analyses. The study on the vertical displacement sensitivity showed that, for the moment-resisting frame buildings, the lower story beams located in the bays containing the removed column were most influential to the progressive collapse mechanism. On the other hand, the progressive collapse mechanism of the dual system building involved many columns including those located far from the removed column. The sensitivity study on the twenty-story case study building showed that the lower story beams played more important roles in resisting progressive collapse than upper story beams. However, it should be pointed out that the accuracy of the research results may depend on the mathematical models for structural materials/members and the statistical data adopted in the sensitivity analysis. Therefore the validity of the analysis results will be enhanced if the analysis is based on more precise material/structural models and more statistical data for design parameters.

The probabilistic approaches of identifying influential parameters to the progressive collapse of steel frame buildings, presented in this paper, can be useful to understand the progressive collapse mechanism and, eventually, to design safer structures against progressive collapse. To do so, it is recommended to understand the force redistribution mechanism and to reduce uncertainty of influential design parameters so that the expected performance of the structure can be achieved with confidence.

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