International Journal of Steel Structures
June 2014, Vol 14, No 2, 1-13
DOI 10.1007/s13296-

International Journal of
STEEL
STRUCTURES
www.springer.com/journal/13296

Design of Special Truss Moment Frames Considering Progressive Collapse

Jinkoo Kim[1] and Jongyeol Park[2]

[1]*Professor, Department of Civil and Architectural Engineering, Sungkyunkwan University, Cheoncheon-dong, Jangan-gu, Suwon, Korea*
[2]*Structural Engineer, eTEC Engineering & Construction, Sinsa-dong, Kangnam-gu, Seoul, Korea*

Abstract

In this study the progressive collapse resisting capacity of the Special Truss Moment Frames (STMF) was investigated. To this end STMF with various span lengths, numbers of story, and lengths of special segment were designed. Their performances against progressive collapse were evaluated based on arbitrary column removal scenario. It was observed that all the model structures designed per the AISC Seismic Provision collapsed as a result of plastic hinge formation at special segment when a column was suddenly removed. A design procedure was developed based on the energy balance concept to prevent progressive collapse. The model structures redesigned using the developed design procedure turned out to remain stable after a column was suddenly removed and satisfy the acceptance criteria of the GSA guidelines.

Keywords: special truss moment frames, progressive collapse, nonlinear analysis, energy based design

1. Introduction

The special truss moment frames (STMF) consist of steel columns and open-web truss girders rigidly connected to form effective seismic load-resisting systems (Itani and Goel, 1991). The truss girder has a special segment designed to behave inelastically under earthquake loads while the other members outside the special segment remain elastic. When a STMF with vierendeel web truss girder is subjected to seismic load, the induced shear force in the girder is resisted primarily by the chord members of the special segment. The ASCE 7-10 (2010) includes the STMF in the seismic force-resisting systems as one of the moment resisting frame systems. The response modification factor of 7 is given for the STMF, which is the second largest value among the R-factors provided for seismic force-resisting systems specified in the specification.

The advantage of using the STMF systems is that the truss girders can be used over longer spans with less amount of steel, and greater overall structural stiffness can be achieved by using deeper girders (Cao and Goel, 2008). Piping and ductwork can be placed through web openings, resulting in better utilization of story space. Another advantage of the system is that the truss girders require relatively simple detailing for moment connections, and the structure can be repairable after being damaged by earthquakes, thus avoiding down-time. The STMF system with X-diagonals was developed by Itani and Goel (1991). Basha and Goel (1994) developed the STMF system without X-diagonals. They also carried out experimental and analytical study of the STMF system with vierendeel middle segment, and found that the hysteretic loops were fuller and nondegrading with saving in steel weight in comparison with the system with X-diagonal configuration. Parra-Montesinos *et al*. (2006) presented the results from the tests of six cantilever double-channel built-up members under reversed cyclic bending, and proposed a new equation for stitch spacing for interconnection of individual channels. Jordan *et al*. (2007) analyzed STMF systems subjected to seismic load, and proposed modified design procedure for special segments introducing pin connections to the chord members. Chao and Goel (2008) employed the plastic design method to design chord members in the special segment. They also presented a direct performance-based plastic design method based on an energy concept and plastic design method which requires no iterative evaluation. Deniz (2009) carried out finite element analysis of STMF and found that the expected shear strength formulation presented in the AISC Seismic Provisions for Structural Steel Buildings is overly conservative. Based on the analysis

Note.-Discussion open until November 1, 2014. This manuscript for this paper was submitted for review and possible publication on May 21, 2013; approved on March 12, 2014.

*Corresponding author
Tel: +82-31-290-7563; Fax: +82-31-290-7570
E-mail: jkim12@skku.edu

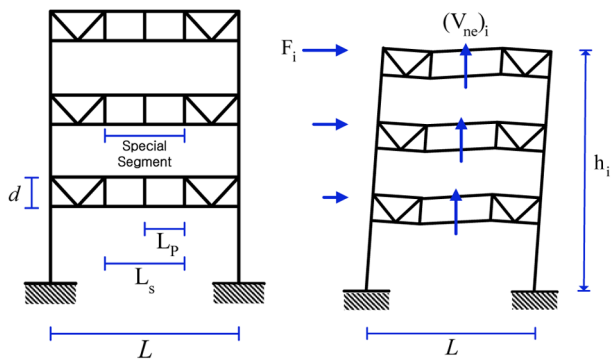
results, he proposed an expected shear strength formula for STMF.

Recently progressive collapse has been considered in the analysis and design of various structures including steel moment resisting frames (Kim and Park, 2008; Kim *et al*., 2009), RC moment frames (Sasani and Kropelnicki 2008), braced frames (Kim *et al*., 2011), and asymmetric tall buildings (Kim and Kong, 2013; Kim and Jung, 2013). This study investigated the progressive collapse resisting capacity of the STMF structures. To this end analysis model structures with vierendeel special segment were designed per the AISC (American Institute of Steel Construction) Seismic Provisions (2010). The design parameters such as the length of special segment, depth of panels, span length, and number of stories were considered in the investigation. The progressive collapse potential of the structures was evaluated based on the arbitrary column loss scenario recommended in the GSA (General Service Administration) guidelines (2003). A design procedure was proposed based on energy-balance principle to prevent progressive collapse of the STMF structures, and the validity of the proposed procedure was evaluated by nonlinear static and dynamic analyses of four analysis model structures.

2. Design of STMF systems

According to the AISC Seismic Provisions (2008), STMF are required to be designed to maintain elastic behavior of the truss members, columns, and connections, except for the members of the special segment that are involved in the formation of the yield mechanism. All members outside the special segment are to be designed for calculated loads by applying the combination of gravity and lateral loads that are necessary to develop the maximum expected nominal shear strength of the special segment. Figure 1 shows the typical configuration and failure mode of the STMF system. The truss girders of the STMF are designed such that the inelastic activity during a seismic event is confined in a special segment near the mid-span. Thus, the chord members at the ends of the special segment are subjected to a combination of axial loads and large inelastic rotations. The yield mechanism of the STMF system consists of the formation of four plastic hinges at the ends of the special segment chords. The AISC Seismic Provisions specify that the length of the special segment shall be between 0.1 and 0.5 times the truss span length. The length-to-depth ratio of any panel in the special segment shall neither exceed 1.5 nor be less than 0.67. The special segment is designed to behave inelastically under seismic load while the remaining members are to behave elastically.

The AISC Seismic Provisions (2008) presents the expected vertical shear strength of the special segment at mid-length, V_{ne}, as follows:



(a) Structural shape (b) Equivalent load and failure mode

Figure 1. Overall shape and behavior of STMF.

$$V_{ne}=\frac{3.75R_yM_{nc}}{L_s}+0.075E_sI\frac{(L-L_s)}{L^3_s}+R_y(P_{nt}-0.3P_{nc})sin\alpha \quad (1)$$

where R_y=yield stress modification factor, M_{nc}=nominal flexural strength of the chord members of the special segment, E_sI=flexural elastic stiffness of the chord members of the special segment, L=span length of the truss, L_s=length of the special segment, center-to-center of supports, P_{nt}=nominal axial tension strength of diagonal members of the special segment, P_{nc}=nominal axial compression strength of diagonal members of the special segment, α=angle of diagonal members with the horizontal members. The first two terms of Eq. (1) were derived based on a study of a vierendeel special segment (Basha and Goel, 1994). One of the assumptions made in the derivation was that the elastic moment at the ends of chord members of the special segment results from vertical translation only, in other words, the effect of end rotation is neglected. This assumption leads to overestimation of the elastic stiffness of the chord members, which in turn results in a higher coefficient, 0.075, in the second term of Eq. (1). This overestimation has a small influence on V_{ne} if the moment of inertia of the chord member is small. However, for heavier chord members the overestimation can be quite large because of their large moment of inertia. Because the members outside the special segment, such as vertical members, diagonal members, connections and columns, are designed based on V_{ne}, any overestimation would result in an overly conservative design of those members. Chao and Goel (2008) presented the modified equation of Eq. (2) to accommodate the above observation.

$$V_{ne}=\frac{3.6R_yM_{nc}}{L_s}+0.036E_sI\frac{L}{L^3_s}+R_y(P_{nt}+0.3P_{nc})sin\alpha \quad (2)$$

The slight modification of the above equation was adapted in the AISC Seismic Provisions 2009 as follows:

$$V_{ne}=\frac{3.6R_yM_{nc}}{L_s}+0.036E_sI\frac{(L-L_s)}{L^3_s}+R_y(P_{nt}+0.3P_{nc})sin\alpha \quad (3)$$

In this study the expected vertical shear strength of the special segment specified in the 2009 Seismic Provision (draft) was used in the seismic design of the STMF structures.

3. Design of Analysis Model Structures

In this section the progressive collapse resisting capacity of a typical STMF structure was investigated. To this end three and five-story STMF structures with different span lengths were designed following the guidelines of the Seismic Provisions. Figure 2 shows the plan shape of the analysis model structures. The exterior frame was separated and the column enclosed in the circle was removed to initiate progressive collapse. Figure 3 shows the side view of the three-story analysis model structure with two different lengths of the special segment. The design dead and live loads of 4.9 and 2.5 kN/m^2, respectively, were used as vertical load, and the seismic load was evaluated based on the spectral acceleration coefficients of S_{DS}=

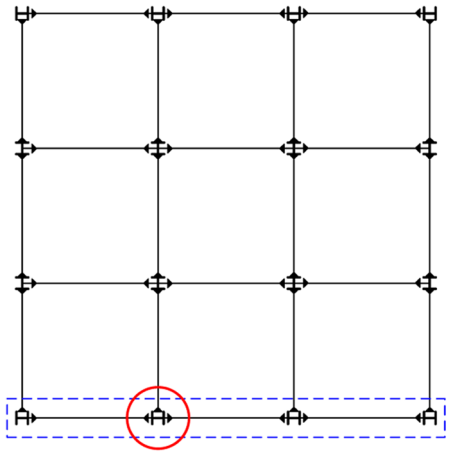
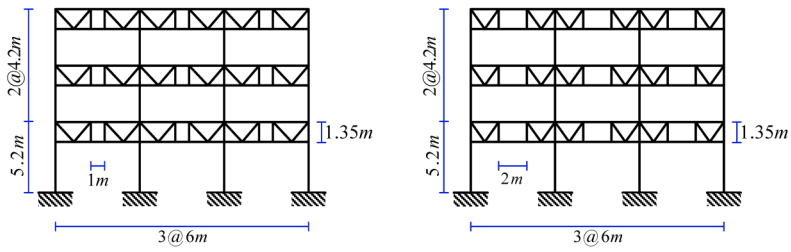


Figure 2. Structural plan and the location of the removed column.



(a) L_s/L=0.16 (b) L_s/L=0.33

Figure 3. 3-story 6 m span model.

Table 1. Member size of model structures (mm)

(a) 3-story 6 m span model (L_s/L=0.33)

Members \ Story	1st	2~3rd
Column (exterior)	H-200×200×8/12	H-150×150×7/10
Column (interior)	H-208×202×10/16	H-175×175×7.5/11
Chord	2L-70×40×8	2L-70×40×8
Chord-SS	2L-110×45×9	2L-90×40×8
Diagonal	2L-45×45×10	2L-45×45×10
Vertical	2L-140×55×11	2L-120×50×10

(b) 5-story 12 m span model (L_s/L=0.33)

	1st	2~3rd	4~5th
Column (exterior)	H-400×408×21/21	H-350×350×12/19	H-300×300×10/15
Column (interior)	H-428×407×20/35	H-414×405×18/28	H-394×398×11/18
Chord	2L-160×70×13	2L-150×70×13	2L-130×60×12
Chord-SS	2L-190×80×16	2L-180×80×16	2L-170×70×14
Diagonal	2L-70×70×12	2L-70×70×12	2L-70×70×12
Vertical	2L-195×80×16	2L-180×80×16	2L-170×70×15
Vertical-SS	2L-175×75×15	2L-165×75×15	2L-140×70×14

Table 2. Steel tonnage of the model structures (kN)

L_s	3-story 6 m span	3-story 9 m span	5-story 12 m span
1 m	508.5	1125	-
2 m	486.3	977.5	2596
2.5 m	528.6	-	-
3 m	-	1152	-
4 m	-	1290	3040
5.7 m	-	-	3738

0.43 and S_{D1}=0.23 with the response modification factor of 7 in the ASCE 7-10 format. The columns were designed with wide flange sections with yield and ultimate strength of 330 and 490 MPa, respectively, and the truss members outside of the special segment were designed with double angle sections with the same strength. The double angle sections in the special segment were designed with steel having the yield and the ultimate strength of 240 and 400 MPa, respectively. The strength ratio of the structural members was set to be 0.9. Table 1 shows the member sizes of the 3-story 6 m span structure and 5-story 12 m span model structure with the special segment to span length ratio of 0.33. Table 2 shows the variation of steel tonnage of the model structures depending on the length of the special segment. It can be observed that the amount of structural steel varies up to 30% depending on the length of the special segment. Except for the structures with 1 m special segment, the steel tonnage generally increased as the length of the special segment increased.

4. Progressive collapse resisting capacity of STMF structures

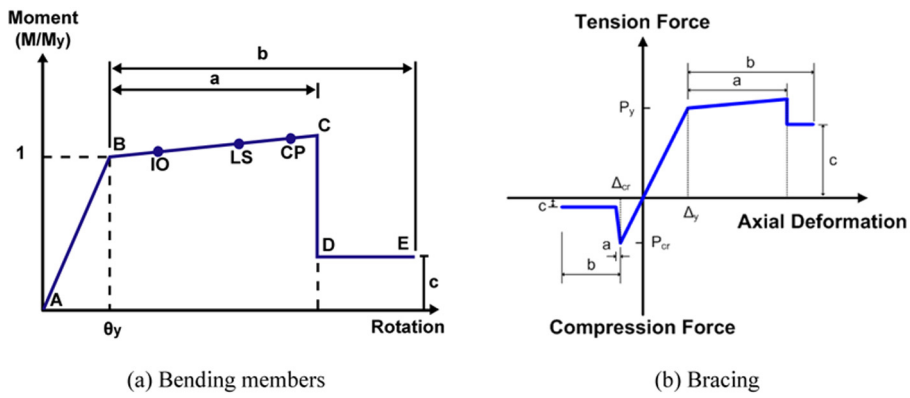
4.1. Analysis methods for progressive collapse

The progressive collapse potential of the model structures were evaluated by arbitrarily removing one of the interior columns and carrying out nonlinear static and dynamic analyses using the program code SAP2000 (2004). The load combination of the GSA Guidelines (2003) for static analysis is 2 (Dead Load+0.25×Live Load). In order to carry out dynamic analysis the member forces of a column are computed before it is removed. Then the column is replaced by the point loads equivalent of its member forces. To simulate the phenomenon that the column is suddenly removed, the column member forces are suddenly removed while the gravity load remains unchanged. More detailed description of the nonlinear analysis procedures for progressive collapse is presented in Kim *et al*. (2009).

For nonlinear analysis of bending members the skeleton curve provided in the FEMA-356 (2003) and shown in Fig. 4(a) was used. The parameters *a*, *b*, and *c* vary depending on the width-thickness ratio of the structural members, and were determined based on the guidelines provided in the Table 5-6 and 5-7 of the FEMA-356. The post-yield stiffness of 3% was generally used for modeling of bending members. For nonlinear analysis of truss and bracing members, the generalized load-deformation curves recommended in the FEMA-274 (1997) and shown in Fig. 4(b) was used, which is based on the phenomenological model proposed by Jain and Goel (1978). The post-yield stiffness of the wide flange sections was assumed to be 3% of the initial stiffness based on the recommendation of the FEMA-356 and that of the double angle sections was assumed to be 10% based on Basha *et al*. (1994). For nonlinear dynamic analysis the damping ratio of 5% was used in all vibration modes.

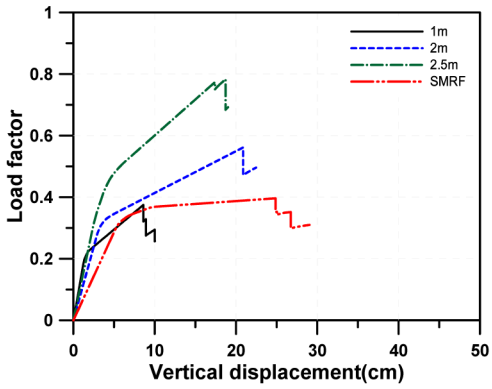
4.2. Effect of design parameters

Figure 5 depicts the pushdown analysis results of the 3-story 6 m span structure with various special segment lengths L_s. For comparison the special moment resisting frame (SMRF) designed with the same dimension using the same loads was also analyzed. It can be observed in Fig. 5(a) that the maximum strengths of the STMF structures are generally higher than that of the SMRF structure and the maximum displacements at failure are smaller than that of the SMRF. The strength of the STMF

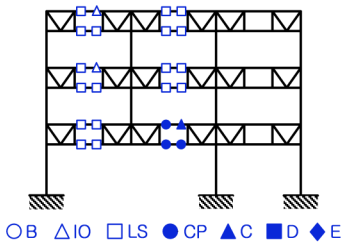


(a) Bending members (b) Bracing

Figure 4. Force-deformation relationship of structural members (IO: Immediate Occupancy, LS: Life Safety, CP: Collapse Prevention).

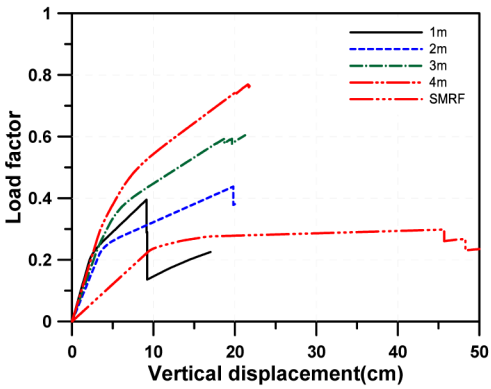


(a) Pushdown curves

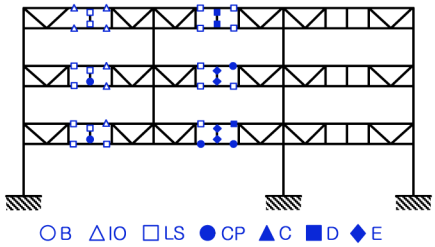


(b) Plastic hinge formation (L_s/L=0.33)

Figure 5. Pushdown analysis results of 3-story 6 m span model structures with various length of special segment.

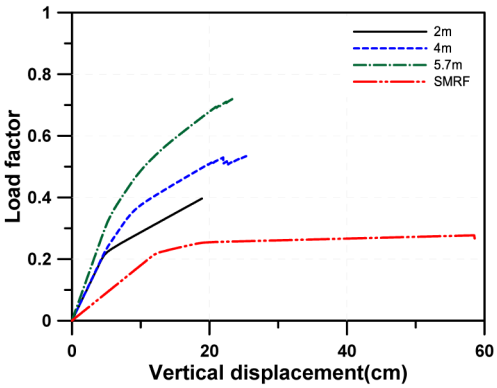


(a) Pushdown curves

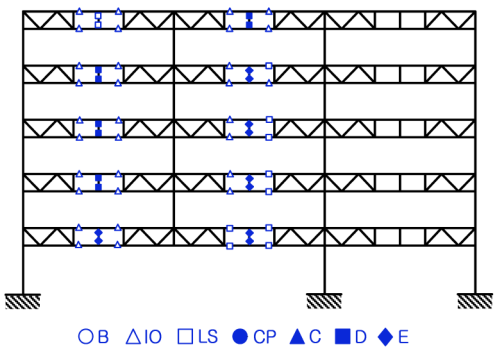


(b) Plastic hinge formation (L_s/L=0.33)

Figure 6. Pushdown analysis results of 3-story 9 m span model structures with various length of special segment.



(a) Pushdown curves

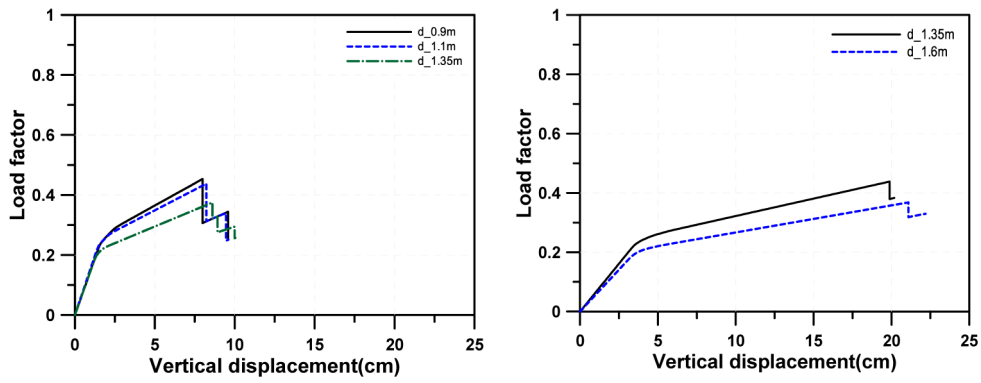


(b) Plastic hinge formation (L_s/L=0.33)

Figure 7. Pushdown analysis results of 5-story 12 m span model structures with various length of special segment.

increased as the length of the special segment increased due to the increased member sizes in the structure with longer special segment. Figure 5(b) shows the plastic hinge formation in the model structures with L_s/L=0.33 at the maximum strength. It can be observed that plastic hinges formed at both ends of the beams in the bays from which a column was removed. In the STMF structure the plastic hinges were concentrated in the special segment and the other parts remained elastic. In comparison with the seismic performance of the SMRF designed with the same loads, the STMF showed higher strength but lower ductility.

Figures 6 and 7 depict the pushdown curves and the plastic hinge formation of the 3-story 9 m span STMF



(a) 3-story 6m span model (L_s/L=0.16) (b) 3-story 9m span model (L_s/L=0.22)

Figure 8. Variation of pushdown curves depending on panel depth.

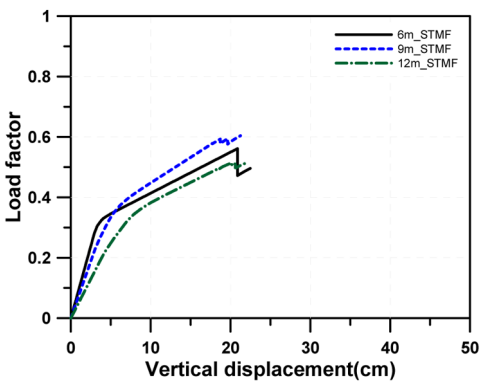


Figure 9. Variation of pushdown curves depending on span length (3-story model (L_s/L=0.33)).

structure and the 5-story 12 m span structure, respectively. It can be observed that the maximum strength increased as the length of the special segment increased, and that plastic hinges formed only at the special segment and the other members behaved elastically. In both cases the SMRF structures showed lowest strength but largest deformation capacity. In all cases the maximum load factors did not reach 1.0, which implies the structures cannot resist the imposed load of 2 (dead load+0.25live load) as specified in the GSA guidelines.

Figure 8 shows the pushdown curves of the three-story STMF structure with varying depth of special segment. The analysis results show that as the depth of the special segment increases and progressive collapse resisting capacity decreases. This is mainly due to the increased lateral load resisting capacity and thus the reduced member size of the chord members in the structure with increased truss depth.

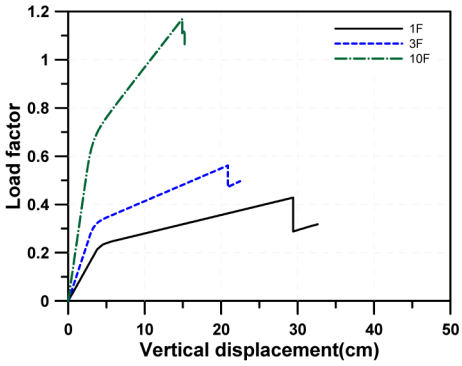
Figure 9 shows the pushdown analysis results of the three-story structures with span length of 6 m, 9 m, and 12 m. In all cases the ratio of the special segment L_s/L is kept 0.33. It can be observed that the strength is largest in the structure with 9 m span length and is smallest in the structure with 12 m span length.

Figure 10 depicts the pushdown curves of the model structures with various stories, where it can be observed that the overall strength against progressive collapse caused by loss of a single column generally increases as the number of story increases. This is due mainly to the increase in redundancy in the structure with larger number of story. It also can be observed that the effect of the number of story depends highly on the span length: The increase in strength is about 36 and 181% as the number of story increases from one to three and one to ten, respectively, in the model structures with 6m span length. The increase reduces to 26 and 53%, respectively, in the structure with 9 m span length. The effect is almost negligible in the structure with 12 m span length.

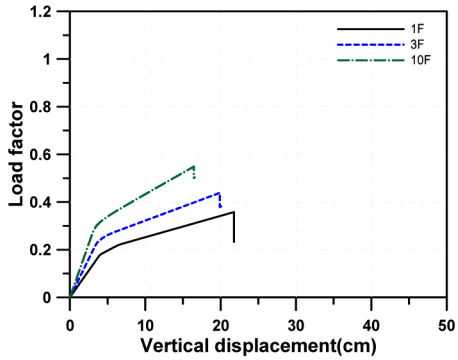
5. Redesign for Preventing Progressive Collapse

The pushdown analyses carried out in the previous section showed that in most cases the vertical strength of the STMF structures designed per the current design code is smaller than required by the GSA guidelines to prevent progressive collapse. In this section a design procedure for SRMF structures is proposed based on energy balance concept to prevent progressive collapse initiated by sudden loss of a column. Similar approach has been applied to prevent progressive collapse of moment resisting frames caused by sudden column loss (Kim and Park, 2008).

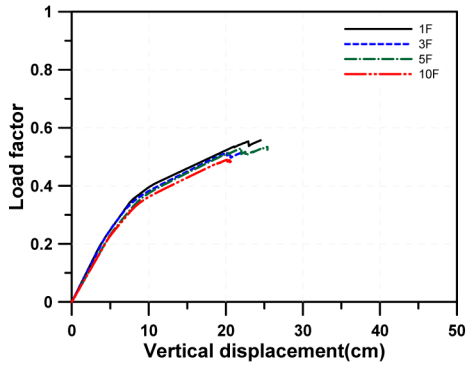
It was observed in the previous section that the inelastic deformation of the STMF structures designed following the AISC Seismic Provisions only occurs in the members of the special segment. In this study a redesign procedure was proposed to enhance the progressive collapse resisting capacity of the STMF structures above the acceptance criterion of the GSA guidelines. Figure 11 illustrates the failure mechanism of the STMF structures subjected to loss of a column, where large plastic deformation occurs in the members located in the special segment. For a STMF structure to remain stable after a column is removed, the internal work of the members subjected to



(a) 6m span model (L_s/L=0.33)



(b) 9m span model (L_s/L=0.22)



(c) 12m span model (L_s/L=0.33)

Figure 10. Variation of pushdown curves of model structures depending on number of stroy.

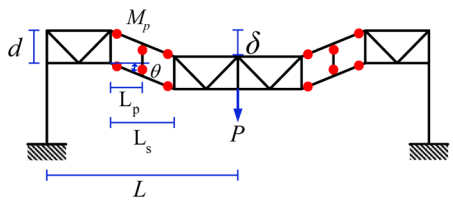


Figure 11. Failure mechanism of STMF subjected to column removal.

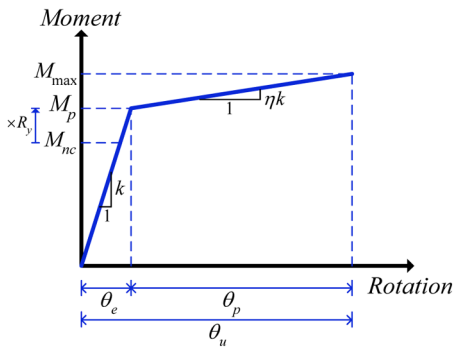


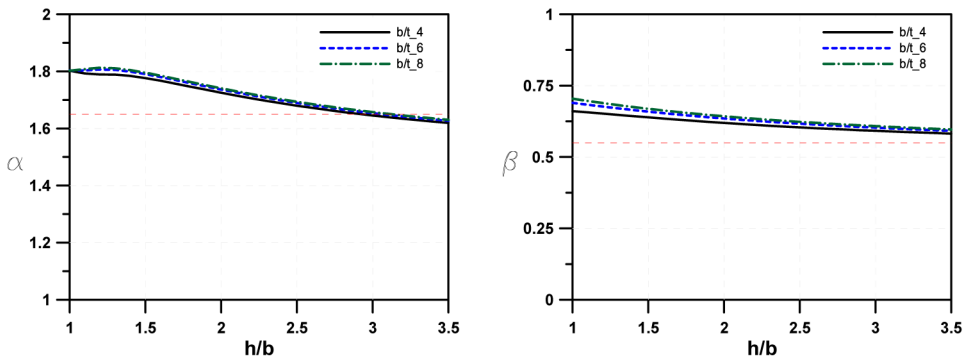
Figure 12. Idealized moment-rotation relationship of members in special segment.

plastic deformation needs to be in equilibrium with the external work done by the removed column. As plastic hinges occur only in the members of the special segment, the required size of the members in the special segment can be obtained from the equilibrium of the internal and external works. Figure 12 shows the moment-rotation relationship of the members in the special segment idealized for design purpose, and the plastic moment of the cord members, M_{pc}, can be obtained as follows:

$$M_{pc} = F_{yc} Z_c = a F_{yc} S_c \tag{4}$$

where F_{yc} is the yield stress of the chord members, S_c and Z_c are the elastic and the plastic section moduli of the chord members, respectively. The shape factor á is the ratio of the plastic and the elastic section moduli. Figure 13(a) shows the parametric study results for the parameters α with respect to the varying h/b (depth/width) of the angle section with three different width thickness ratios (b/t). It can be observed that α decreases almost monotonically from 1.8 to 1.65 as h/d varies from 1 to 3. In this study lower bound value of 1.65 was used for α to derive conservative solution for the required section modulus of the special segment members to prevent progressive collapse. The plastic moment of the vertical members in the special segment can be computed as follows:

$$M_{pv} = F_{yv} Z_v = a F_{yv} S_v = \gamma M_{pc} \tag{5}$$



(a) α (b) β

Figure 13. Parametric studies for design coefficients.

where F_{yv} is the yield stress of the vertical members, S_v and Z_v are the elastic and the plastic section moduli of the vertical members in the special segment, respectively, and γ is the ratio of the plastic moment of the vertical and the chord members as follows:

$$\gamma = \frac{M_{pv}}{M_{pc}} \tag{6}$$

In the bi-linearly idealized moment-rotation relationship of the members in the special segment, shown in Fig. 12, the yield rotations of the chord and the vertical elements are obtained as follows:

$$\theta_{ec} = \frac{M_{pc}L_p}{6E_sI_c}, \quad \theta_{ev}\frac{M_{pv}d}{6E_sI_v} \tag{7}$$

where E_s is the elastic modulus, I_c and I_v are the second moments of inertia of the chord and the vertical members, respectively, L_p is the length of the special segment, and d is the depth of the special segment panel. The limit state for member rotation was set to be 0.035rad following the GSA guidelines. The moments of inertia of the chord and the vertical members in the special segment are represented as follows:

$$I_c = S_c\beta h_c, \quad I_c = S_c\beta h_v \tag{8}$$

where βh is the depth of the centroid. The variation of the parameter β as a function of *h/b* of the angle section is depicted in Fig. 13(b), where it can be observed that β decreases monotonically from about 0.7 to 0.6 as *h/b* increases from 1.0 to 3.5. In this study the lower bound value of 0.55 was used for β to induce conservative results. Based on the above simplification, the energy balance equation of the internal and the external work is formulated as follows:

$$N\times\left(\frac{M_{pc}\theta_{ec}}{2}+\frac{(2M_{pc}+\eta k_c\theta_{pc})\theta_{pc}}{2}\right)+N_v$$

$$N_v\times\left(\frac{M_{pc}\theta_{ev}}{2}+\frac{(2M_{pv}+\eta k_v\theta_{pv})\theta_{pv}}{2}\right)=P\times\delta \tag{9}$$

where

$$k_c = \frac{6E_sI_c}{L_p}, \quad k_v = \frac{6E_sI_v}{d} \tag{10}$$

The left hand side of Eq. 9 represents the internal work done by the member force and the deformation of the

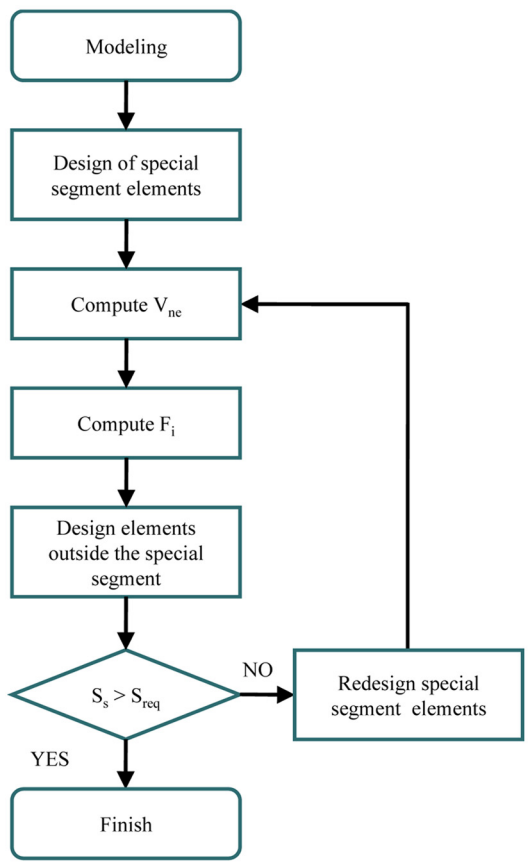


Figure 14. Flow chart of design procedure for preventing progressive collapse of STMF.

Table 3. Sectional dimension (mm) and section modulus (cm^3) of members in special segment before and after redesign

(a) 1-story 6m span model (L_s/L=0.33)

Members	Story	Original (S_i)	S_{req}	Redesigned (S_f)
Chord	1	2L-80×40×9 (25.6)	67.4	2L-120×55×11 (70.7)

(b) 3-story 9m span model (L_s/L=0.22)

Members	Story	Original (S_i)	S_{req}	Redesigned (S_f)
Chord-SS	1	2L-120×60×11 (71.9)	162.3	2L-175×65×12 (161.8)
	2~3	2L-110×50×10 (54.1)	121.9	2L-165×65×10 (122.2)

(c) 3-story 12m span model (L_s/L=0.33)

Members	Story	Original (S_i)	S_{req}	Redesigned (S_f)
Chord	1	2L-190×80×16 (255.9)	420.3	2L-231×90×18 (422.3)
	2~3	2L-170×75×15 (193)	316.8	2L-205×85×17 (316.2)
Vertical	1	2L-180×75×15 (215.2)	230.1	2L-187×75×15 (231.3)
	2~3	2L-140×70×14 (123.6)	132.1	2L-145×70×14 (132.1)

(d) 10-story 12m span model (L_s/L=0.33)

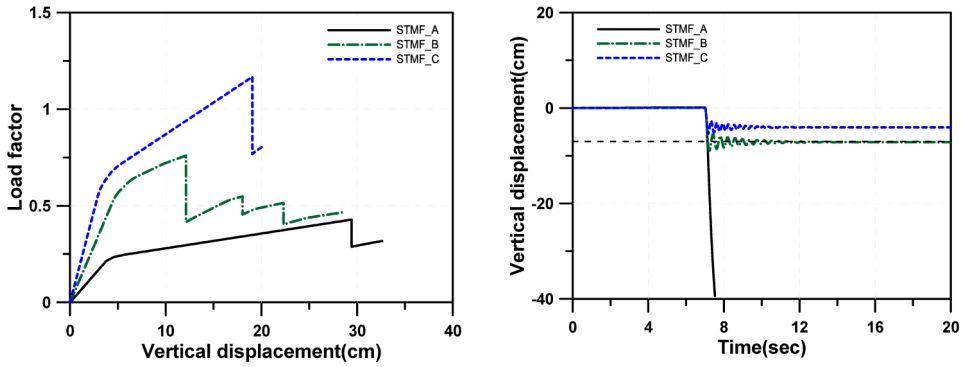
Members	Story	Original (S_i)	S_{req}	Redesigned (S_f)
Chord	1	2L-200×85×17 (301.6)	444.7	2L-238×90×18 (446.8)
	2~3	2L-185×80×16 (243.3)	358.7	2L-220×85×17 (361.4)
	4~5	2L-180×75×15 (215.2)	317.2	2L-213×80×16 (317.9)
	6~7	2L-165×70×14 (169.1)	249.2	2L-195×75×15 (250.4)
	8~9	2L-160×70×14 (159.4)	235.1	2L-189×75×15 (236.1)
	10	2L-160×70×14 (159.4)	235.1	2L-189×75×15 (236.1)
Vertical	1	2L-195×75×15 (250.4)	386.5	2L-221×90×18 (388.3)
	2~3	2L-180×70×14 (199.4)	307.7	2L-202×85×17 (307.4)
	4~5	2L-170×70×14 (178.9)	276.1	2L-198×80×16 (276.8)
	6~7	2L-150×70×14 (140.9)	217.6	2L-181×75×15 (217.4)
	8~9	2L-145×70×13 (123.6)	190.7	2L-176×70×14 (191.1)
	10	2L-120×60×12 (77.8)	120.1	2L-145×65×13 (121.9)

elements in the special segment, and the right hand side corresponds to the external work done by the force supported by the removed column, P, and the vertical displacement, d, at the beam-column joint from which the column was removed. N_c and N_v are the number of plastic hinges formed in the chord and the vertical members in the special segment, and θ_{pc} and θ_{pv} are the plastic rotation at the chord and the vertical members, respectively. The post yield stiffness η was assumed to be 10% of the initial stiffness. Based on the above equations the section moduli of the chord and the vertical members in the special segment required to satisfy the energy balance equation, $S_{c(\mathrm{req})}$ and $S_{v(\mathrm{req})}$, respectively, are derived as follows for the given depths of the chord and the vertical members, h_c and h_v, respectively:

$$S_{c(req)} = \frac{2PL_s\theta_u}{N_c\left[aF_{yc}\left(1.8\theta_u - \frac{0.15aF_{yc}L_P}{\beta h_c E_s}\right) + \frac{0.6\beta h_c E_s \theta_u^2}{L_p}\right] + \gamma N_v\left[aF_{yv}\left(1.80_u - \frac{0.15aF_{yv}d}{\beta h_v E_s}\right) + \frac{0.6\beta h_c E_s \theta_u^2 F_{yc}}{dF_{yv}}\right]} \tag{11}$$

$$S_{v(req)} = \gamma S_{c(req)} \frac{F_{yc}}{F_{yv}} \tag{12}$$

To prevent progressive collapse of the STMF structures caused by sudden column loss, the sectional moduli of the members in the special segment need to be larger than those derived above. Therefore after a STMF is designed based on the current design code, the above procedure needs to be applied before finalization of design. Once the member sizes of the special segment are increased, the other members also need to be redesigned so that plastic hinges form only at the special segment. The



(a) Pushdown curves (b) Time histories of vertical displacement

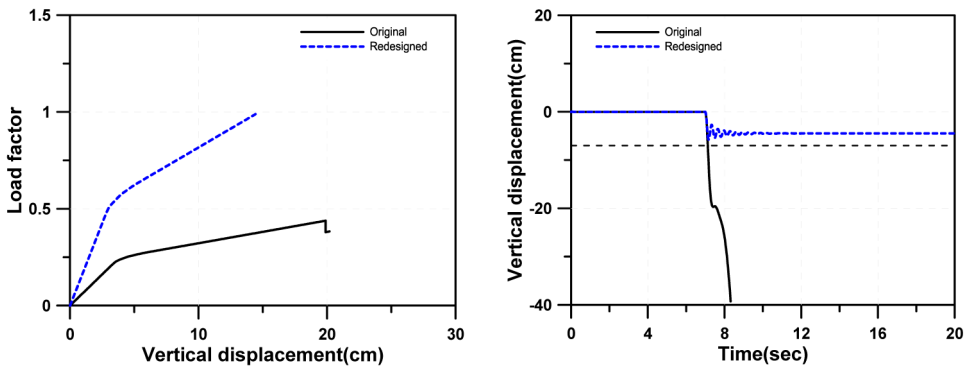
Figure 15. Analysis results of 1-story 6m span model (L_s/L=0.33).

(a) Pushdown analysis (b) Pushover analysis

Figure 16. Plastic hinge formation in the 1-story 6 m span model with only special segments redesigned.

(a) Pushdown analysis (b) Pushover analysis

Figure 17. Plastic hinge formation in the 1-story 6 m span model with all elements redesigned.

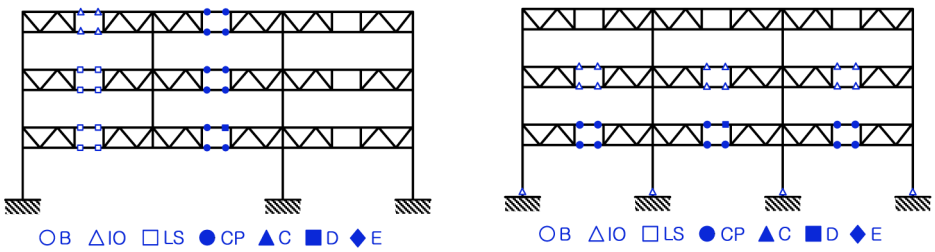


(a) Pushdown curves (b) Time histories of vertical displacement

Figure 18. Analysis results of 3-story 9 m span model (L_s/L=0.22).

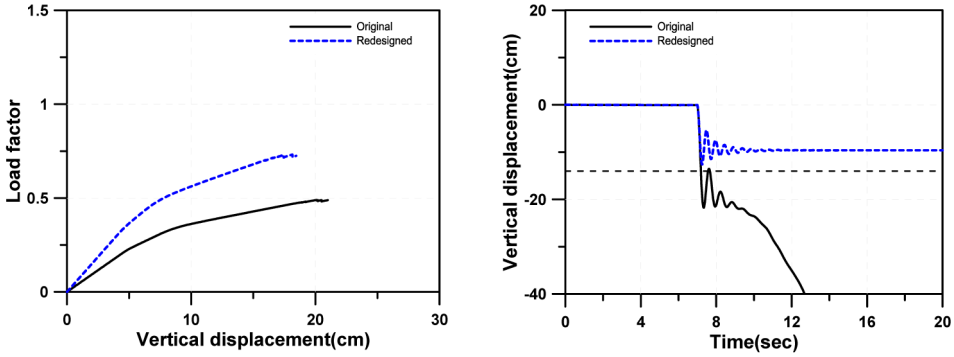
modified design procedure to ensure safety against progressive collapse is depicted in Fig. 14.

The validity of the proposed design procedure to prevent progressive collapse was investigated by analyzing four STMF structures before and after redesign. The initial and the final member sizes of the special segment designed without and with considering progressive collapse, respectively, are presented in Table 3. Figure 15 shows the nonlinear static and dynamic analysis results of the single story STMF structure with 6m span length (L_s/L=0.33) subjected to sudden loss of one of the interior columns. The results of the structure before redesign (Original) and after redesign were compared. Also compared are the results of the structure with only the members in the special segment redesigned (Redesigned-SS only). The pushdown analysis results show that the maximum load factor of the original structure designed per the AISC Seismic Provision is less than 0.5, well



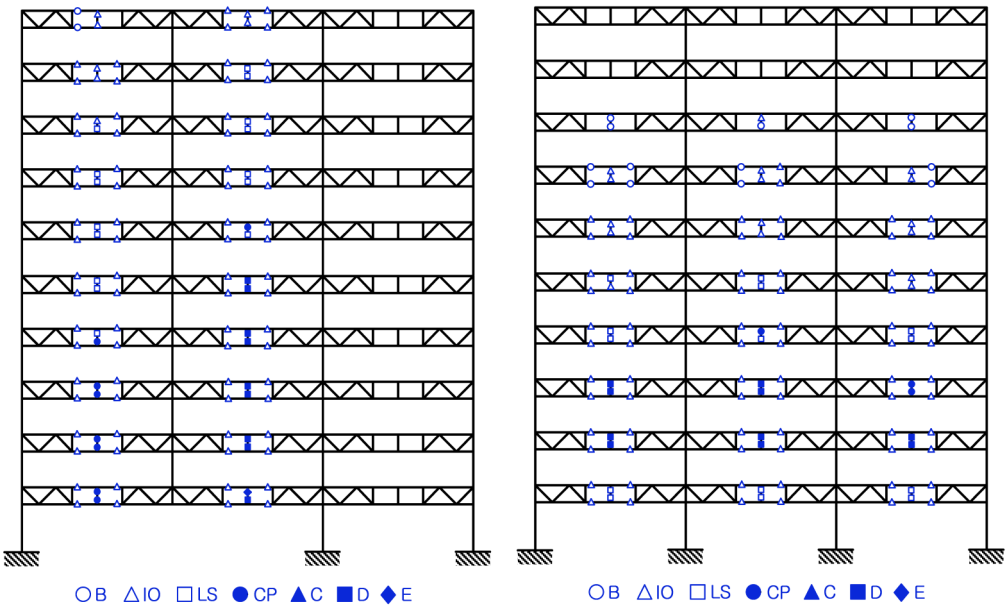
(a) Pushdown analysis (b) Pushover analysis

Figure 19. Plastic hinge formation in the 3-story 9 m span model.



(a) Pushdown curves (b) Time-histories of vertical displacement

Figure 20. Analysis results of 10-story 12 m span model (L_S/L=0.33).



(a) Pushdown analysis (b) Pushover analysis

Figure 21. Plastic hinge formation in the 10-story 12m span model.

below the required value of 1.0. The nonlinear time history analysis results show that the vertical displacement is unbounded when the column is suddenly removed. The maximum load factor of the structure with only the member sizes of the special segment redesigned considering progressive collapse reached about 0.75 and the structure remained stable around the vertical displacement specified as limit state in the GSA guidelines after sudden removal of the column. The structure with complete redesign showed maximum load factor higher than 1.0 and remained stable at the vertical displacement above the limit state. Figures 16 and 17 show the plastic hinge

formation in the 1-story model structure obtained from pushdown and pushover analyses. Two sets of analysis results were presented; the one for the structure with its special segment redesigned (Fig. 16) and the other for the structure with all other members redesigned to meet the AISC Seismic Provisions (Fig. 17). It can be observed that a few plastic hinges formed outside of the special segment in the structure with only its special segment redesigned for progressive collapse. However in the structure with all members redesigned following the proposed procedure, plastic hinges formed only in the special segment, which conforms to the basic philosophy of STMF structures. Figures 18 to 21 depict the nonlinear static and dynamic analysis results of the 3- and 10-story model structures. In every case the structure designed based only on the AISC Seismic Provisions turned out to collapse by sudden loss of an interior column. However the structures designed following the proposed procedure turned out to remain stable at vertical displacements smaller than the limit states specified in the GSA guidelines. It was also observed that plastic hinges formed only in the special segment as required by the Seismic Provisions either when they were subjected to seismic load or exposed to sudden column loss.

6. Summary

In this study the progressive collapse resisting capacity of Special Truss Moment Frames (STMF) was investigated based on the arbitrary column removing scenario. As analysis models, STMF with various span lengths, numbers of story, and lengths of special segment were designed and their performances were compared using nonlinear static and dynamic analyses.

Nonlinear static pushdown analysis showed that when a column was removed all the plastic hinges formed at special segment and the other parts of the structures remained elastic. The vertical strength of the model structures increased as the length of the special segment increased and the depth of the truss panel decreased. According to the nonlinear dynamic analysis results of the model structures subjected to sudden loss of a column, all the model structures designed per the ATSC Seismic Provisions collapsed due to failure of special segment. A closed form formula was derived to obtain the required section moduli of the members in the special segment to prevent progressive collapse based on the energy balance concept. The remaining elements were resized based on the AISC Seismic Provisions to ensure plastic hinge formation only in the special segment. The model structures redesigned using the developed design procedure turned out to satisfy the acceptance criteria of the GSA guidelines to prevent progressive collapse. The nonlinear static pushover analysis of the redesigned structures showed that plastic hinges formed only in the special segment as required by the Seismic Provisions.

Acknowledgment

This research was financially supported by the Samsung Research Fund of Sungkyunkwan University.

References

AISC (2005). Seismic provisions for structural steel building. *AISC-341-05*, American Institute of Steel Construction, Chicago, Illinois

AISC (2010). Seismic provisions for structural steel building (draft). *AISC-341-10*, American Institute of Steel Construction, Chicago, Illinois

ASCE 7-10 (2010). *Minimum design loads for buildings and other structures*. American Society of Civil Engineers, New York

Basha, H. and Goel, S. C. (1994). Seismic resistant truss moment frames with ductile vierendeel segment, *Report No. UMCEE 94-29*, Dept. of Civ. & Env. Engrg., Univ. of Michigan, Ann Arbor, Michigan.

Chao, S. H. and Goel, S. C. (2008). "Performance-based plastic design of special truss moment frames." *Engineering Journal,* pp. 127-150

Deniz, O. H. (2009). *A numerical study on special truss moment frames*. PhD Thesis, Middle East Technical University

FEMA (1997). NEHRP Guidelines for the seismic rehabilitation of building. *Report No. FEMA-273*, Federal Emergency Management Agency, Washington, D.C.

FEMA (2000). Prestandard and commentary for the seismic rehabilitation of buildings. *FEMA-356*, Federal Emergency Management Agency, Washington, D.C.

Goel, S. C. and Itani, A. M. (1994). "Seismic-resistant special truss moment frames." *Journal of Structural Engineering*, ASCE, 120(6), pp. 1781-1797.

GSA (2003). *Progressive collapse analysis and design guidelines for new federal office buildings and major modernization project*. The U.S. General Service Administrations.

Itani, A. and Goel, S. C. (1991). Earthquake resistant design of open web framing systems. *Research Report No. UMCE 91-21*, University of Michigan, Dept. Civil and Environmental Eng., Ann Arbor, MI.

Jain, A. K. and Goel, S. C. (1978). Inelastic cyclic behavior of bracing members and seismic response of braced frames of different proportions. *Report No. UMEE78R3*, Department of Civil Engineering, University of Michigan, Ann Arbor, Michigan.

Jordan, S. J., Elfass, S., and Pekcan, G. (2007). "Seismic response investigation of special truss moment frame subassemblies under quasi-static loading." *Proc. 2007 Earthquake Engineering Symposium for Young Researchers*, Seattle, Washington.

Kim, J. and Jung, M. (2013). "Progressive collapse resisting capacity of tilted building structures." *Struct. Design Tall Spec. Build.*, 22, pp. 1359-1375.

Kim, T., Kim, J., and Park, J. (2009). "Investigation of progressive collapse resisting capability of steel moment resisting frames using pushdown analysis." *J. Performance*

of Const. Facilities, 23(5), pp. 327-335.

Kim, J. and Kong, J. (2013). "Progressive collapse behavior of rotor-type diagrid buildings." *Struct. Design Tall Spec. Build.*, 22, pp. 1199-1214.

Kim, J., Lee, Y., and Choi, H. (2011). "Progressive collapse resisting capacity of braced frames." *The Structural Design of Tall and Special Buildings,* 20(2), pp. 257-270.

Kim, J. and Park, J. (2008). "Design of steel moment frames considering progressive collapse." *Steel and Composite Structures*, 8(1), pp. 85-98.

Parra-Montesinos, G. J., Goel, S. C., and Kim, K. Y. (2006). "Behavior of steel double-channel built-up chords of special truss moment frames under reversed cyclic bending." *J. of Struct. Eng.*, 132(9), pp. 1343-1351.

SAP2000 (2004). *Structural analysis program*. Computers and Structures, Berkeley, CA

Sasani, M. and Kropelnicki, J. (2008). "Progressive collapse analysis of an RC structure." *The Structural Design of Tall and Special Buildings*, 17(4), pp. 757-771.