Paper 24

Proceedings of the Eighth International Conference
on Engineering Computational Technology,
B.H.V. Topping, (Editor),
Civil-Comp Press, Stirlingshire, Scotland

The Design of Special Truss Moment Frames Against Progressive Collapse

H.K. Kang, J.Y. Park and J.K. Kim
Department of Architectural Engineering
Sungkyunkwan University, Suwon, Korea

Abstract

In this study the progressive collapse resisting capacity of the special truss moment frames (STMF) was investigated. As analysis models, STMF with various span lengths, numbers of storeys, and lengths of special segment were designed and their performance compared. It was observed that all the model structures designed as per the ATSC seismic provision collapsed as a result of plastic hinge formation at the special segment when a column was suddenly removed. A design procedure was developed to prevent progressive collapse based on the energy balance concept. The model structures redesigned using the developed design procedure turned out to remain stable after a column was suddenly removed and satisfies the acceptance criteria of the GSA guidelines.

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

1 Introduction

This study investigated the progressive collapse resisting capacity of the special truss moment frames (STMF) structures. To this end analysis model structures with vierendeel special segment were designed per the AISC (American Institute of Steel Construction) Seismic Provisions. 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. 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.

The AISC Seismic provisions 2009 (draft) presents the expected vertical shear strength of the special segment at mid-length, V_{ne}, as follows:

$$V_{ne} = \frac{3.6R_y M_{nc}}{L_s} + 0.036 E_s I \frac{(L - L_s)}{L_s^{\ 3}} + R_y (P_{nt} + 0.3P_{nc}) \sin \alpha \qquad (1)$$

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.

3 Design of analysis model structures

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. Fig. 1 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 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. Fig. 3 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:

$$M_{pc} = F_{yc} Z_c = \alpha F_{yc} S_c \qquad (4)$$

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 α is the ratio of the plastic and the elastic section moduli.

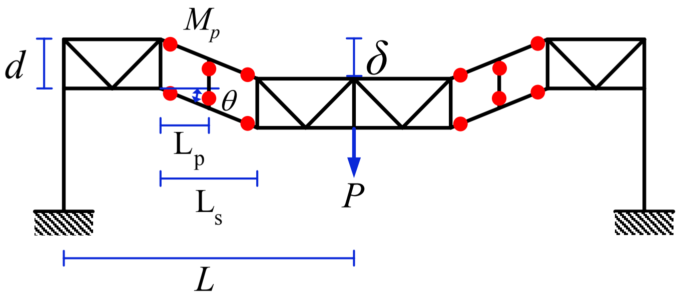


Fig. 1 Failure mechanism of STMF subjected to column removal

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) showed 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:

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

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:

$$\theta_{ec} = \frac{M_{pc} L_p}{6 E_s I_c}, \qquad \theta_{ev} = \frac{M_{pv} d}{6 E_s I_v} \tag{7}$$

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:

$$I_c = S_c \beta h_c, \qquad I_v = S_v \beta h_v \tag{8}$$

where βh is the depth of the centroid. The variation of the parameter β as a function of *h/b* of the angle section showed 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:

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

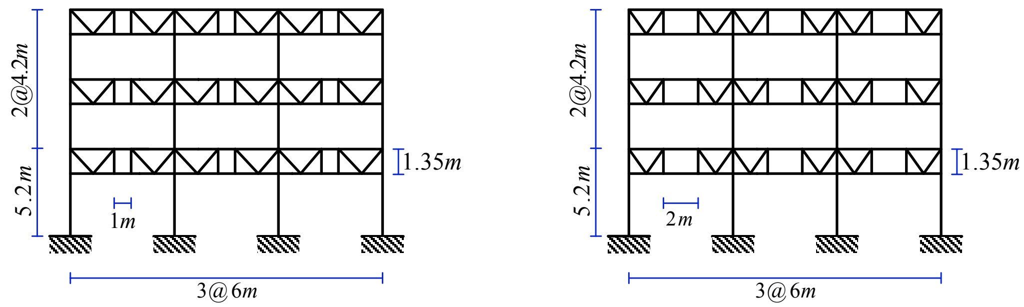
$$k_c = \frac{6E_s I_c}{L_p}, \qquad k_v = \frac{6E_s I_v}{d} \tag{10}$$

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

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

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

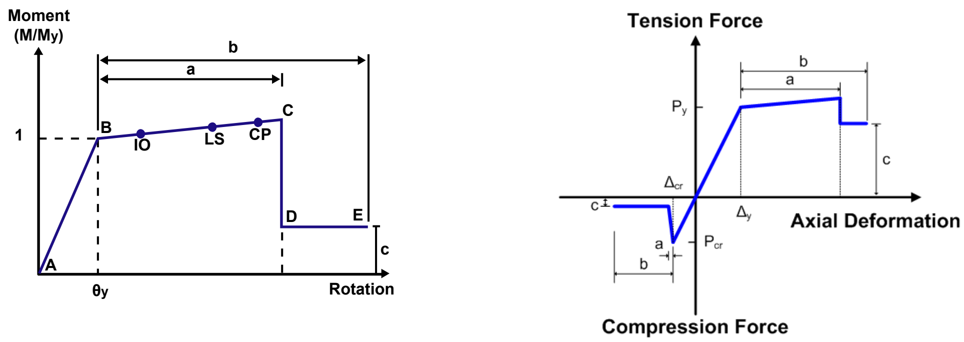
To prevent progressive collapse of the STMF structures caused by sudden column loss, the sectional moduli of the members in the special segment need to be larger than those derived above. Therefore after a STMF is designed based on the current design code, the above procedure needs to be applied before 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.



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

Fig. 2 3-story 6m span model

As analysis model structures three-story STMF structures with different span lengths were designed following the guidelines of the Seismic Provisions. Fig. 2 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 4.9kN/m^2 and 2.5kN/m^2, respectively, were used as vertical load, and the seismic load was evaluated based on the spectral acceleration coefficients of S_{DS}=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 ultimate strength of 490 MPa, and the truss members outside of the special segment were designed with double angle sections with the same ultimate strength. The double angle sections in the special segment were designed to have the ultimate strength of 400 MPa. Fig. 3 shows the force-deformation relationship of structural members (IO: Immediate Occupancy, LS: Life Safety, CP: Collapse Prevention).

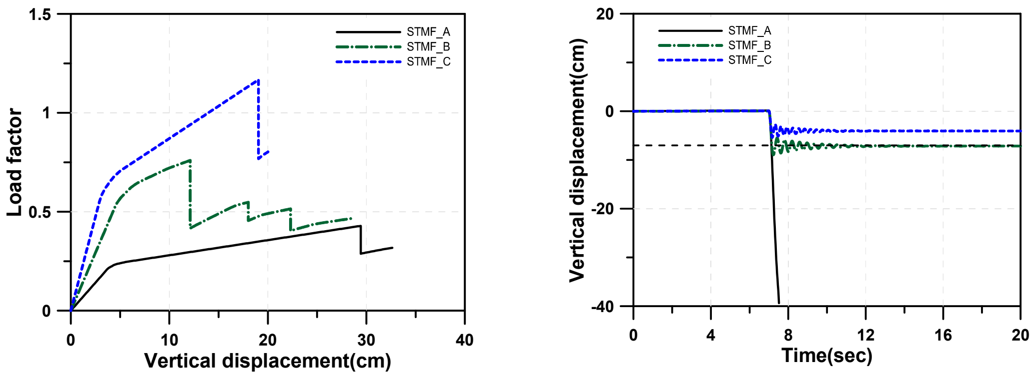


(a) Bending members (b) Bracing

Fig. 3 Force-deformation relationship of structural members (IO: Immediate Occupancy, LS: Life Safety, CP: Collapse Prevention)

4 Analysis of model structures

The validity of the proposed design procedure to prevent progressive collapse was investigated by analyzing STMF structures. Fig. 4 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 designed with conventional (Original) and the proposed method (Redesigned) were compared. 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 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. Fig. 5 shows the plastic hinge formation in the 1-story model structure obtained from pushdown and pushover analyses. It can be observed that 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. Fig. 6 depicts the Plastic hinge formation in the 3-story 9m span model. 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.

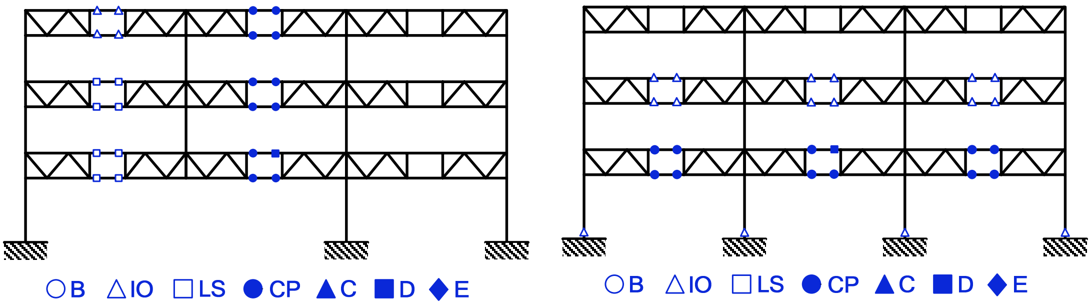


(a) Pushdown curves (b) Time histories of vertical displacement

Fig. 4 Analysis results of 1-story 6m span model ($L_s/L=0.33$)

(a) Pushdown analysis (b) Pushover analysis

Fig. 5 Plastic hinge formation in the 1-story 6m span model with all elements redesigned with proposed method



(a) Pushdown analysis (b) Pushover analysis

Fig. 6 Plastic hinge formation in the 3-story 9m span model

5 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 storeys, and lengths of special segment were designed and their performances were compared using nonlinear static and dynamic analyses.

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.

Acknowledgement

This research was financially supported by a grant (Code# ’09 R&D A01) funded by the Ministry of Land, Transport and Maritime Affairs of Korean government.

References

[1] AISC. Seismic Provisions for Structural Steel Building (Draft). AISC-341-10, American Institute of Steel Construction, Chicago, Illinois, 2010

[2] ASCE 7-10, "Minimum Design Loads for Buildings and Other Structures", American Society of Civil Engineers, New York, 2010.

[3] GSA, Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Project, The U.S. General Service Administrations, 2003

[4] SAP2000, Structural Analysis Program, Computers and Structures, Berkeley, 2004