Nonlinear dynamic behavior of steel framed roof structure with self-centering members under extreme transient wind load

Linjia Bai, Yunfeng Zhang *

Department of Civil and Environmental Engineering, University of Maryland, College Park, MD 20742, USA

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A B S T R A C T

Collapse of roof could cause severe economic loss and poses safety risk to residents in the building. This paper presents a nonlinear dynamic analysis of steel framed roof structures with force limiting devices under combined static and transient wind loading. Two types of force limiting devices – one with self-centering behavior and the other exhibiting bilinear-like hysteresis are examined for roof collapse prevention. A nonlinear dynamic analysis that accounts for both material and geometrical nonlinearities was carried out for this simulation study. Two types of steel framed roof structures – a K-series steel joist and an arch truss are selected as the prototype roof frame in this study. It is found that installing force limiting devices at intentionally weakened zone of the prototype steel framed roof structures helps mitigating the displacement demand of the roof frame structure under transient up-lift wind pressure and thus reduce the dynamic collapse risk. Furthermore, the force limiting devices with self-centering behavior minimizes the residual deflection of the roof structures after the wind event.

1. Introduction

Steel structural framings is a popular structural form to cover the large roof space of gymnasiums, industrial facilities and transportation terminals, with potential use as shelter structures for a disastrous event such as hurricanes. However, under high winds, either part of the roof enclosure or the entire roof structure can be lifted off a building, particularly for low sloped roofs subject to wind-induced suction force. Collapse of roof could cause severe economic loss and poses safety risk to residents in the building.

Transient change in wind pressure may happen during a severe windstorm such as downburst and impose dynamic uplift load on low sloped roof structures. Such dynamic uplift load may cause failure of certain members in roof frames which may initiate the collapse of a steel framed roof structure as a result of load redistribution. The failure or rupture of members in a steel framed roof structure, may be dynamic in nature [31,14] and thus a nonlinear time history analysis is required to faithfully capture the dynamic collapse behavior of roof frames under transient wind loads. The collapse of a steel roof frame structure can be initiated by the buckling of a few members, as a result of load redistribution causing a subsequent progressive overstress condition in other members and thus its load carrying capacity is usually limited by the failure of first member or set of members to fail.

In an attempt to alter the brittle collapse behavior of steel roof frame structures under extreme wind loading, force limiting devices (FLD) can be used for robustness enhancement. These devices are fitted to critical compression members, and designed to provide a purely plasto-elastic behavior with a long plateau of member ductility (e.g., as exhibited in buckling-restrained braces). Since collapse is initiated by only a few critical members, the use of force limiting devices can be limited to a small selection of the most critical compressive chord or web diagonal members. The feasibility of using force limiting devices for controlling the behavior of space truss was validated in a research conducted by ElSheikh [8]. The principle behind using these devices is to introduce artificial ductility in truss compression members, which would otherwise possess brittle post-buckling characteristics involving a loss of both stability and strength upon buckling. However, large residual deformations in conventional force limiting devices after strong loading events can make the structure appear unsafe to occupants, impair the structural response to a subsequent loading and significantly increase the cost of post-event repair or replacement.

Recognizing the importance of controlling the residual deformation, self-centering seismic resisting system has recently been attracting considerable attention from the community (e.g., [23,3,35]). A flag-shaped hysteresis loop is typical of such self-centering systems, which is able to reduce (or even eliminate) residual structural deformation. Dolce et al. [6] tested Nitinol-based devices with full re-centering and good energy dissipation capabilities. Zhu and Zhang [35] studied a special type of bracing member termed...
self-centering friction damping brace (SFDB) which exhibits a flag-shaped hysteresis loop and has a potential to be used for force limiting devices with self-centering behavior. In the SFDB, the self-centering ability is realized using super elastic SMA wires while its enhanced energy dissipation capacity is achieved through friction.

This paper presents the numerical study results of dynamic collapse risk mitigation of steel framed roof structures by installing force limiting devices under combined gravity load and transient wind loads. Both material and geometric nonlinearities are considered in the nonlinear dynamic collapse analysis. The formulations are 2-dimensional in this study. In the nonlinear analysis, dynamic effects associated with sudden loss of member capacity due to buckling and lateral inertial forces at the mid-length point of a buckling member are considered. Two numerical examples – one from a standard Steel Joist Institute (SJI) K series joist and the other on an arch tubular truss were used to demonstrate the effectiveness of the force limiting devices in mitigating the dynamic collapse risk of steel framed roof structures under transient wind load.

2. Nonlinear dynamic analysis procedure

To accurately simulate the dynamic collapse of steel roof framing structures subjected to combined gravity and transient wind load, non-linear time history analysis was performed in this study. The finite element model, that considers dynamic member buckling, yielding of tension members, and geometric nonlinearity due to large deflection of the roof structures undergoing collapse, is established using the OpenSees simulation platform [17].

A progressive collapse refers to a structural failure that is initiated by localized structural damage and subsequently develops, as a chain reaction, into a failure involving a major portion of the structural system [7]. Several mechanisms might contribute or lead to the progressive collapse of steel roof frame structures including: buckling of compression member, yielding of a tension member, and nodal instability. The collapse of a steel frame structure can be initiated by the buckling of a few members [1], and its load carrying capacity is usually limited by the failure of first member or set of members to fail.

In this study, a 2-dimensional force-based element with corotational formulation and fiber discretization of the cross section was adopted for the simulation of buckling, post buckling, and hysteretic responses of members in steel roof frames. In this force-based corotational element, both material and geometric nonlinearities are considered.

The formulation of force-based elements is based on interpolation functions for the internal force variation [30]. Neuenhofer and Filippou [21] formulated a force-based element for geometric nonlinear analysis of plane frame structures, with linear constitutive relations and small rotations. Following that, de Souza [26] extended the material nonlinear force-based element proposed by Neuenhofer and Filippou [20] to include geometric nonlinearity through deriving the transverse displacements from the curvatures using Lagrangian polynomial interpolation. The adopted kinematics is based on the assumption of moderately large deformations along the element; rigid body displacements and rotations can be arbitrarily large. However, at a price of further subdividing the structural member into smaller elements, large deformation problems can also be solved.

In the analysis of frame elements, the material nonlinearity is considered by integrating the material stress-strain relations defined for each fiber over the section area, which is commonly referred to as “fiber discretization”. Distributed plasticity is obtained by integration of the section force-deformation response over the member length in the corotating frame of reference. As shear effects are neglected, a uniaxial stress-strain relation is employed at the material point. The effect of spreading of plastic deformation along the member axis is thus considered in this study.

The following equation of motion is used for nonlinear time history analysis,

$$\mathbf{M}\ddot{\mathbf{U}}(t) + \mathbf{C}\dot{\mathbf{U}}(t) + \mathbf{R}(\mathbf{U}(t)) = \mathbf{P}_f(t)$$  \hspace{1cm} (1)

where the first term is the acceleration-dependent inertial force vector, \(\mathbf{R}\) is the displacement-dependent restoring-force vector. \(\mathbf{P}_f(t)\) is the external applied-force vector.

The standard approach for dynamic collapse analysis, as well as other nonlinear structural dynamic problems, is to time discretize the governing equations by Newmark time integration then solve them via the Newton-Raphson algorithm [28]. In this study, the Newmark constant average acceleration method (\(\beta = 0.25\) and \(\gamma = 0.50\)) is employed. This requires that iteration must be performed at each time step in order to satisfy equilibrium. Also, the incremental stiffness matrix must be formed and triangularized at each iteration or at selective points in time.

3. Numerical analysis model

This section describes the components used in the numerical simulation of the dynamic collapse of steel roof frames under combined transient wind loading and gravity load. A finite element analysis software program –OpenSees [16] is used in this study.

3.1. Analytical model for dynamic member buckling

Modeling of the buckling behavior of steel bracing members has been examined by a number researchers (e.g., [11,33,12]). An initial imperfection has to be assigned to the axially loaded member to simulate its buckling behavior. However, few studies have considered the dynamic buckling effect of a compression member caused by the mid-point-inertia-mass (mid-mass)-induced lateral force at its mid-length [31].

The prototype member is a steel tube (O.D = 4.5 in (114.3 mm), I.D = 3.83 in (97.28 mm)) with a material yield strength of 30 ksi (206 MPa). The member length is set to be 80 in. (2.32 m). To capture the inertia effect on member dynamic buckling, a lumped mass which represents half the mass of the member is assigned to the middle-length point of the member following a procedure described by Tada and Suito [31]. A parametric study has been conducted and the results revealed that a finer meshed model has very little effect on the global collapse behavior of the roofs. Therefore, a single mass lumped at mid-length was used in all cases. The end conditions are specified as pin-roller supported, as shown in Fig. 1. Thus, the member was divided into two inelastic beam-column elements with an initial camber of 0.1% at the mid-node of the member in the finite element model, as shown in Fig. 1.

In this element, fiber discretization was used and the force components were obtained by integration over the cross-section. The tubular section of the member was discretized into two layers of fibers in the radial direction and 20 fibers in the circumferential direction.

![Fig. 1. Configuration of brace model considering mid-length mass.](image-url)
direction respectively. As shear effects are neglected, a uniaxial stress-strain relation is employed at the material point. The material properties for the simulation model are based on the Menegotto-Pinto model for steel with extensions for kinematic and isotropic hardening [9] with the following values: Young's modulus = 29,000 ksi (200 GPa), yield stress = 30 ksi (206.8 MPa), and strain-hardening ratio = 0.005.

Fig. 2 shows the load–displacement \((P–\Delta)\) relations of the prototype member under monotonic and cyclic load, \(P\), respectively. Due to the initial camber introduced at the mid-length point, the member is subjected to second-order bending effect. At a critical value of \(\Delta\), once the second order bending moment at the mid-length point reaches the member’s plastic moment capacity, a plastic hinge with spread plasticity would form there. Further pushing the member in the longitudinal direction would result in the spreading of the plastic deformation near mid-length region and reduction in load. Fig. 2a shows the load–displacement \((P–\Delta)\) relations of the prototype member using three different analysis methods: static analysis using displacement control method, static analysis using arc-length method [24], and dynamic analysis. It is seen that the dynamic load–displacement for the forced deformation oscillates around the static relationship after buckling, due to inertia force of the mid-mass associated with sudden buckling-induced lateral movement. Both static analyses using two control methods cannot simulate such oscillation phenomenon because inertia mass effect is not considered. It is also seen that the critical load values for the arc-length case and dynamic analysis case are close to each other, evidencing the effectiveness of the dynamic analysis method in modeling the buckling behavior of the compression member. Clearly, this two-element model with initial camber and mid-mass is able to capture the dynamic buckling, post-buckling stiffness degradation, tension strength increase due to strain hardening, and as such this model can be used in the subsequent collapse analysis.

A parametric study was performed to examine the adequacy of dividing the member into two elements to model the buckling behavior of a compression member and the results are presented in Fig. 2b for monotonic displacement and Fig. 2c for cyclic displacement respectively. The initial camber of the member is set as 0.1% of the member length. Not much difference was observed between the three cases with 2, 4 and 8 subdivided elements for the compression member respectively. Therefore, a coarse scheme with two inelastic beam elements is sufficient for the accurate representation of the overall hysteretic behavior of the compression member concerned.

3.2. Force limiting device: mechanism and modeling

In an effort to alter the brittle collapse behavior of steel roof frame structures under extreme wind loading, force limiting de-
vices (FLDs) can be used for robustness enhancement. Also, to minimize the residual deformation of the roof structure associated with conventional FLD exhibiting bilinear hysteresis, self-centering members similar to the one developed by Zhu and Zhang [35] provides a promising FLD alternative for use in steel roof frame structures.

FLDs with a long plateau of member ductility can be used for collapse risk mitigation of roof frame structures by fitting them to substitute critical compression members. The fundamental concept behind FLD is to restrain low-order member buckling modes, creating full and stable hysteretic loops under tension–compression cyclic load. For example, with the use of such FLDs as buckling restrained braces [32], member buckling can be held off till a fairly large compressive strain, without significant strength degradation.

Two types of force limiting devices – conventional FLD exhibiting bilinear hysteresis and self-centering (SC) FLD with re-centering behavior after unloading are examined in this study for roof collapse prevention. In numerical modeling of FLD, a simple, one-dimensional, pin-ended truss element is used to have the appropriate uniaxial force–displacement properties. The hysteresis loop of the conventional FLD model calibrated with the experimental data reported by Clark et al. [5] are shown in Fig. 3, in comparison with that of the corresponding buckled bracing model and SC FLD.

Flag-shaped hysteretic model, as shown in Fig. 3, has been widely used to represent the cyclic loading behavior of self-centering system for sake of its simplicity (e.g. [3,4,25,35]). Two key parameters that define this flag-shaped hysteretic model are post-yield stiffness ratio $\alpha$ and energy dissipation factor $\beta$ that relates to the energy dissipation capacity of the system. For example, its lower bound (i.e., $\beta = 0$) describes a piecewise nonlinear elastic system while the upper bound (i.e., $\beta = 1.0$) corresponds to a self-centering system with greatest possible energy dissipation capacity. For a given system with known initial stiffness and yield strength, the flag-shaped hysteretic model can be completely defined with these two parameters – $\alpha$ and $\beta$. In this study, three different $\beta$ values were considered: 0.2, 0.5, and 0.8. The $\alpha$ value is set as 0.005 in this study.

The hysteretic model that represents the load–displacement relationship of the conventional FLD is also shown in Fig. 3. This is based on the Menegotto-Pinto model for steel with extensions for kinematic and isotropic hardening [9] with a strain-hardening ratio of 0.001.

Typically, a compromise has to be struck between the improvement to structural behavior associated with these devices and the cost addition caused by their use. Therefore, the use of FLDs is limited to a small selection of the most critical compression chord members in this study. This includes a strategy for selectively replacing key compressive members in steel roof structures with FLDs to prevent collapse or at least alter the collapse sequence to a favorable collapse mechanism. Simple linear finite element analysis of the original steel roof frame structure without FLDs was conducted to determine the internal forces in all members. The locations of the FLDs are chosen based on the axial force from this analysis. Several other plausible pressure distribution cases were also conducted and found these selected places are also fine in order to improve the structural collapse resistance behavior compared with the original frame without FLDs. FLDs work as fuse members, where the deformation of the structure would concentrate; and because its strength is weakened compared to regular member, FLD would first be activated under many load cases.

A structural weakening concept was employed in this study to improve the structural behavior under dynamic wind loading. This concept is similar to selective weakening retrofit in earthquake engineering [22]. Note that the yield force of FLD was designed to be reduced to 30% of its original strength (i.e., yield stress times cross-sectional area), which is lower than the critical load (i.e., buckling force) of a compression member with imperfection (around 37% of its original strength). Although 30% is lower than the critical load 37%, it is worth noting that after member buckles, both strength and stiffness degrade rapidly while self-centering member still maintain a stable load carrying capacity and stiffness after ‘yielding’. This is the essential distinction between using weakened self-centering members and regular buckled member

![Fig. 3. Hysteresis model of buckled member, conventional FLD and SC FLD.](image)

![Fig. 4. Transient wind speed time history used in this study: (a) wind speed time history; (b) frequency spectrum.](image)
even though the critical load for the buckled member is slightly greater. This distinction leads to the favorable behavior of steel framed structure with self-centering members even under quasistatic load like the one considered in this study.

The purpose of member weakening in the roof framing is to make the selected weakened member as a fuse element where favorable ductile damage of the overall structure is concentrated. This concentrated structural weakening strategy could also bring other benefits such as ease in damage inspection and rapid structural repair with member replacement post extreme events. Strengthening critical members only would not improve the structure collapse behavior because doing so will increase the forces on their neighboring members and cause them buckle instead. To strength all structural members is clearly not economical.

3.3. Transient wind load profile

Despite the fact that extensive research has been carried out on structures subjected to stationary wind loads, the ultimate behavior of steel roof structures under high-intensity transient wind loads, such as downburst, is less well understood. The effect of downburst on structures has recently gaining attention due to the collapse of lightweight steel structures such as transmission towers and roof structures [27,18,13]. A downburst is created by a column of sinking air that, after hitting ground level, spreads out in all directions and is capable of producing damaging straight-line winds, often producing damage similar to, but distinguishable from, that caused by tornadoes. Microburst was first recognized by Fujita in 1978. Microburst can occur anywhere, and a number of metropolitan areas in the US have experienced microburst with gust wind up to 130 mph (209.2 kph) in the past (e.g., [10,15]). Another collapse of steel joist roof structures reported at Dallas was also attributed to microburst-induced uplift force during a thunderstorm event in February 2001 (Nelson et al., 2006). Additionally, a hurricane could also cause transient wind loading on roof. In a hurricane, gusts of wind can be expected to be 25–50% higher than the sustained wind velocity.

The wind speed can be expressed as the sum of a mean speed and a fluctuating part:

$$U(z, t) = \bar{U}(z) + u(z, t)$$

where mean speed $\bar{U}(z)$ is a function of height $z$; fluctuating speed $u(z, t)$ is a function of height $z$ and time $t$. According to Chay et al. [2], modified Osegueda and Bowles/Vicroy (OBV) model is effective to simulate the mean wind $\bar{U}(z)$. However, this is beyond the scope of this study.

A well-known downburst record is that which struck the Andrews Air Force Base (AAFB) near Washington, DC, on August 1, 1983 [10]. For simplification, the original AABF record is used as the mean wind and the fluctuating wind was simulated by superimposing a white noise time series. The fluctuation part of the wind speed is simulated by adding a Gaussian white noise series to the AABF record. The peak value of this fluctuation part is set to be equal to 10% of the peak of mean wind speed, that is, 7 m/s. To study the effect of transient wind load on the collapse behavior of steel roof frame structures, a typical wind speed history is given in Fig. 4 along with its corresponding frequency spectrum.
Clearly, the highest frequency content of the wind load is below 0.5 Hz and thus no dynamic effect from the wind load is induced on the prototype structures with their fundamental frequency greater than 3.8 Hz.

4. Case study I: steel joist roof frame

A common construction method in the United States, particularly for light commercial and public buildings, is the use of lightweight metal roof decking supported by steel bar joists. This type of roof system is economical for gravity load carrying; however, when subjected to extreme transient wind load such as induced by downburst, it could potentially be damaged [19].

A typical roof joist—26K9 (K series) in the SJI catalog [29] is selected for this study and full lateral support is assumed. A 2-dimensional analysis was performed with no out-of-plane displacements. The selected 26K9 joist is approximately 50 ft (15.24 m) in span length, 26 in. (0.66 m) in depth, and spaced at 12.5 ft (3.81 m) on center and spanned between column bays in a 50 ft by 50 ft grid (15.24 m by 15.24 m). The design of chord sections for the K-Series joist is based on a yield strength of 50 ksi (345 MPa) while the design of web sections for the K-Series joist is based on a yield strength of 36 ksi (250 MPa). The configuration of the joist and its cross section are shown in Fig. 5. Both the top and bottom chord are made from two $2 \times 2 \times 3/16$ in. (5.08 × 5.08 × 0.476 cm) steel angles back to back while webs are $15/16$ in. (2.38 cm) diameter rounds.

The dead loads applied to the structure include the self-weight of the joist and roofing materials, which were applied to the nodes of top chords. The roofing also includes building integrated photovoltaic panels (BIPVs) and the weight of each 1.028-m by 0.629-m BIPV panel is calculated as 30 kg from data supplied by a leading BIPV manufacturer. The self-weight of the joist was calculated as 11.2 lb/ft (163.2 kN/m) and the remaining roof covering load was 9.70 psf (0.464 kPa). So the total dead load was 10.6 psf (0.508 kPa). The dead loads for internal joists spaced 12.5-ft (3.8 m) apart is then converted into concentrated load applied to the top chord nodes. The transient wind load was also applied as concentrated loads at the nodes of upper chords following a spatial load profile given in Fig. 5.

4.1. Numerical simulation

The finite element model established in the OpenSees consists of a total of 43 corotational nonlinear beam-column elements. The top and bottom chords are modeled as a continuous member with no moment release at the chord nodes, while the web diagonals in the joist are pin-connected to both the top and bottom chords. The left support of the joist is assumed to be pin supported while the right is a roller support. The lumped mass assigned to the...
top chord nodes are determined based on the corresponding dead loads. The natural frequencies of this steel joist roof frame are calculated from the finite element model as: 3.85, 14.1, 26.3, 30.3, 43.5, 58.8, and 71.4 Hz for the first seven vibration modes. For the cases with FLDs, very small changes in modal frequencies are observed as following: 3.83, 13.70, 25.64, 29.41, 41.67, 55.56, and 71.4 Hz. The viscous damping ratio of the frame is assumed to be 2%. The time interval is set as 0.02 s in the numerical simulation.

The “collapse” in this study is defined to be the point when the mid-span deflection exceeds 2% of the roof span length. In the process of dynamic collapse, some members of the structure can be assumed not to fail in buckling or fracture while those critical members will buckle or yield and go into the inelastic range. Those critical members were modeled by subdividing into two beam-column elements, as illustrated in Fig. 1. A middle node is essential to faithfully capture the large displacement at the middle length point of these members. As shown in Fig. 5a, member U2U4, U1L1, USL5 and mid-bottom chords (L2L3 and L3L4) are subdivide into two elements respectively. A 0.1% camber of the original member length was assumed for their initial imperfections.

4.2. Results and discussions

Fig. 6 shows the displacement time histories of node L3, which has the largest deflection values and thus is used as a reference point. For the case of regular steel joist frame without FLD (labeled as “No FLD”), the roof frame failed due to excessive deflection. It is

![Structure deformation](image)

**Fig. 8.** Displaced shape of steel joist roof frame under transient wind load.
seen in Fig. 6a that there is a sharp upward turn in the displacement response curve, indicating occurrence of member failure slightly before 228 s when the peak wind pressure is reached. After checking the member response, members U1L1 and U4L5 were found to buckle at that moment and thus cause the sudden upward motion. It is also seen in Fig. 6 that after members U1L1 and U4L5 were replaced by FLDs, the roof structure did not collapse because the FLDs work as fuse members resulting in reduced axial forces in neighboring members. It is worth noting that strength reduction has been intentionally made in the FLD members to fulfill its function as fuse member. The yield strength of FLDs including both conventional FLD and SC FLDs was reduced to be 30% of the original member’s axial yield strength based on the results of several trial simulations. This reduced value is close to the critical buckling load of member U1L1 and U4L5. Consequently, this leads to reduced load demand on their neighboring members and thus prevents the failure of these members. However, due to its bilinear-like hysteresis, a residual displacement as large as 50-mm after the wind event is observed in both Fig. 6a and b for the roof frame with conventional FLDs. In order to minimize the residual displacement, the SC FLDs are used and the corresponding results are shown in Fig. 6b for different $\beta$ values. It is also seen that the energy dissipation factor $\beta$ has very little effect on the displacement response during the time period of peak wind speed (approximately from 220 to 250 s).

Fig. 7a shows the hysteresis loops of those FLDs at the location of element U1L1 of the steel joist roof structure. The use of FLDs for replacing those critical compression members eliminates the sudden upward movement (upward overshoot) of the steel joist structure due to member buckling and thus enhance its collapse resistance. Clearly, incorporating FLDs into the steel roof framing can enhance its robustness in resisting roof collapse under transient wind loading. For the Cases with SC FLDs, the residual deformations of member U1L1 were zero after unloading and thus minimize the global residual displacement of the roof structure. It is also seen in Fig. 6a that the $\beta$ factor only affects the unloading force and has no effect on the global displacement response of the steel joist roof. The global hysteresis loop of support reaction force and mid-span node displacement obtained from five simulations are as shown in Fig. 7b respectively. The use of FLDs dramatically decreases the maximum deflection on node L3. An analysis was conducted to obtain the critical uplift wind pressure values for each case. The critical uplift wind pressure value refers to the lowest peak wind pressure value that leads to the roof collapse. The critical wind pressures 1.501 kPa for the original structure (labeled as No FLD) and 1.71 kPa for Cases with FLDs. For the cases with SC FLDs, their critical wind pressures are the same as those with conventional FLD. It can be seen from Fig. 8 that failure happens in the uplift phase when the upward deflection exceeds the pre-specified value for roof collapse. During the unloading phase, the SC FLDs may cause different axial force values on their neighboring members compared with the conventional FLDs, but this would not cause them to buckle because the axial force is lower than their critical loads. It is seen that the critical wind pressure value increases about 14% after using FLDs.

Fig. 8 shows the variation of configuration shape of the steel joist roof structure during the collapse process. It’s clear that the buckling of the critical compression member U1L1 triggered the roof collapse and the use of SC FLD prevents this from happening. Other benefits of SC FLDs include no residual displacement of the roof structure after the wind event.
5. Case study II: arch truss roof frame

An arch truss is also selected as the prototype roof frame structure, as shown in Fig. 9. The top and bottom chords are steel tubes with an outer diameter of 2.38 in. (6.05 cm) and an inner diameter of 1.94 in. (4.9 cm). The web diagonals in the truss web are steel tubes with an outer diameter of 1.66 in. (4.22 cm) and an inner diameter of 1.28 in. (3.25 cm).

5.1. Numerical simulation

The finite element model of this arch truss consists of 81 corotational beam-column elements. The web diagonals were modeled as pin-connected by releasing the moment at their ends. The top and bottom chords of the arch truss are assumed to be continuous over their entire length with no moment release at the element nodes. The supports at node L1 and L8 are pin supported. The location of FLDs are indicated in Fig. 9.

The dead loads applied to the arch truss nodes include its self-weight and the gravity load due to roof coverings such as BIPV glass roof. The self-weight of the arch truss is calculated to be 20 lbs/ft (0.29 kN/m) and the additional load from roof covering is 11.23 psf (0.47 kPa). The total dead load is thus equal to 11.4 psf (0.55 kPa). The dead loads for the internal joists spaced 12.5-ft (3.8 m) apart are then converted into concentrated load applied to the top chord nodes. The external wind load was applied to the joints of the upper chords of the arch truss as concentrated load.

Table 1

Critical wind pressure values for two types of steel framed roof structures (unit = kPa).

<table>
<thead>
<tr>
<th></th>
<th>FLD, ( \beta = 0.2 )</th>
<th>FLD, ( \beta = 0.5 )</th>
<th>FLD, ( \beta = 0.8 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type I structure</td>
<td>1.50</td>
<td>1.71</td>
<td>1.71</td>
</tr>
<tr>
<td>Type II structure</td>
<td>1.57</td>
<td>2.22</td>
<td>2.22</td>
</tr>
</tbody>
</table>

Fig. 10. Arch truss response under transient wind load: global response.

Fig. 11. Arch truss response under transient wind load: member hysteresis.

Fig. 12. Arch truss response to transient wind load: global hysteresis.
loads in the normal direction to the roof surface following a spatial load profile given in Fig. 9. The lumped mass assigned to the top chord nodes are determined based on the corresponding dead loads. The natural frequencies of this Type II roof frame are calculated from the finite element model as: 3.94, 5.98, 8.58, 8.63, 11.9, 13.7 and 16.4 Hz for the first seven vibration modes respectively. For the cases with FLDs, only slight changes in modal frequencies are observed as following: 3.85, 5.86, 8.39, 8.27, 11.54, 13.05, and 15.47 Hz. The viscous damping ratio of the frame is assumed to be 2%. The time interval is set as 0.02 s in the numerical simulation.

In order to model members' buckling behavior, initial imperfections were introduced into all chord members and diagonal web members as shown in Fig. 9. The critical loads of vertical web members were found to be very large due to their short length. Since the actual axial forces on the vertical members are much
smaller than their critical load, buckling of vertical web members are not modeled in this study. The initial camber at the mid-length point of the member with initial imperfection is set to be 0.1% of their original length.

5.2. Results and discussions

Unlike what is observed from the steel joist roof frame, the arch truss roof structure didn’t exhibit the phenomenon of “over-shooting” in deflection after peak wind pressure is reached. This can be explained by the special configuration shape of the arch truss roof frame. For the arch truss structure, all outer and inner chords were subjected to tension like a catenary during the uplift process and most of the members are in tension. Immediately after the peak wind pressure is reached, the arch truss drops down instead of kept going up like that seen for steel joist roof frame. Vertical displacement at node L4 was selected as the failure index for this arch truss roof frame because of its maximum vertical displacement in the entire structure. The results of L4 displacement are shown in Fig. 10. For Case No FLD (i.e., the original structure without FLDs), the structure failed immediately at 200 s. After close examination, it is found that this failure can be attributed to the buckling of member U3U4. In contrast, the case of Conventional FLD did not fail after 200 second and still stable after the peak wind. However, a fairly large residual deflection of 160 mm at node L4 is observed due to the bilinear like hysteresis associated with the conventional FLDs. In order to reduce the residual displacement, SC FLD members are used and the corresponding responses are plotted in Fig. 10b. The residual displacements for all \( \beta \) values were reduced to around 10 mm. The \( \beta \) value has no effect on the residual displacement of the arch truss. However, different unloading behaviors are observed in the arch truss with SC FLDs of different \( \beta \) value. The cases with higher \( \beta \) values take longer time to return to its original shape.

Fig. 11a shows that the \( \beta \) value has no effect on the residual deformation of element U3U4 but did show distinct unloading behavior. This can also be seen in the critical wind pressure values for two types of steel framed roof structures listed in Table 1. Fig. 11b shows that the regular FLD will introduce residual deformation on element U5U6 which causes the global residual displacement seen in Fig. 10. Using SC FLDs will eliminate the residual displacements. Fig. 12 shows the global hysteresis of the steel arch truss structure. The enclosure area is larger when the ratio \( \beta \) is increased which implies more energy dissipation.

An analysis was conducted to obtain the critical uplift wind pressure values for each case. The critical wind pressures 1.57 kPa for Case No FLD and 2.215 kPa for Cases with FLDs. For the cases with SC FLDs, their critical wind pressures are the same as those of the conventional FLD cases. It is seen that the critical wind pressure value increases about 41\% by using FLDs to replace the critical compression members.

Fig. 13 shows the variation of configuration shape of the arch truss during the collapse process. It is seen that the buckling of member U3U4 triggers the start of the failure and using SC FLDs prevent the collapse and the entire arch truss re-center to its original position after unloading.

Energy dissipation does not play much role here. As explained above, the advantage for using self-centering members is that their strength and stiffness is maintained (thus stable behavior) after reaching ‘yield’ point while for buckled members, both strength and stiffness degrade rapidly (thus unstable behavior). This is the main reason why the self-centering frames has improved collapse resistance behavior compared to the case without FLDs, even under quasi-static load like the one considered in this study.

6. Conclusions

This research presents the results of nonlinear dynamic analysis of steel frame roof structures subjected to transient wind load. To accurately simulate the dynamic collapse of steel framed roof structure subjected to transient wind loading, dynamic effects associated with sudden loss of member capacity due to buckling and lateral inertial forces associated with mass lumped at the mid-length point of a buckled member are taken into account in this simulation study. In order to simulation member buckling and associated strength degradation and large deformation, a 2-dimensional force-based element with corotational formulation and fiber discretization of the cross section was adopted, which accounts for both material and geometric nonlinearities.

Two types of steel framed roof structures – a K-series steel joist and an arch truss are selected as the prototype roof frame in this study. In an attempt to alter the brittle collapse behavior of steel roof frame structures under extreme wind loading, force limiting devices (FLD) can be used for robustness enhancement. These devices are fitted to substitute critical compression members, and designed to provide a high level of member ductility. Two types of force limiting devices – conventional FLD exhibiting bilinear hysteresis and self-centering (SC) FLD with re-centering behavior after unloading are examined in this study for roof collapse prevention. Preliminary study was conducted to select the critical compression members which were weakened and replaced with FLDs to serve as fuse members. Through weakening of the steel frame roof by using FLDs with reduced yield strength, damage in the roof frame can be concentrated to the pre-selected fuse members while the entire roof structure’s integrity is retain during extreme transient wind event. Findings from this numerical study include:

1. The two-element model for buckling member with initial camber and mid-length lumped mass is able to capture the dynamic buckling, post-buckling stiffness degradation, tension strength increase due to strain hardening, and as such this model can be used in the dynamic buckling analysis.

2. Installing FLDs at intentionally weakened locations in steel framed structures helps mitigating the deformation demand of the roof frame structure under transient up-lift wind pressure. Unlike the original critical compression member susceptible to buckling, the FLD could maintain its strength and exhibit ductile load behavior. When the sudden increase of wind pressure leads to overshoot of roof truss structures in its upward motion, which causes buckling of certain members in the original roof structure, steel roof frame structures with FLDs can withstand the transient wind load without collapse. The strengths of FLDs usually have to be reduced (compared to the critical load of its original substituted member) to prevent the buckling of their neighboring members.

3. The use of conventional FLDs exhibiting bilinear-like hysteresis results in residual deflections in the steel frame roof structure. To minimize residual deflection, self-centering FLDs with flag-shaped hysteresis can be employed. It is found that the energy dissipation factor \( \beta \) has very little effect on the response behavior of steel frame roof structures under transient wind load in terms of peak deflection and residual deflection.

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