A DAMAGE-PLASTICITY APPROACH FOR DETERIORATION MODELING OF STEEL COMPONENTS

T.N. Do¹ and F.C. Filippou²

ABSTRACT

This paper adopts a recent development in beam and column element formulation based on resultant plasticity and damage mechanics for large-scale simulations and collapse assessment of steel structures. The strength and stiffness deterioration in the element response is described by a damage variable that is continuously updated based on the hysteretic energy dissipation and the maximum or minimum deformation. The new damage-plasticity beam and column element models are deployed in a pilot study of an 8-story 3-bay steel special moment-resisting frame that is subjected to far-field ground motions with scaled intensity up to collapse. The study examines the location and the magnitude of the model's damage variables within the structure and concludes that the damage distribution is indicative of the collapse mechanisms and closely related to both the local element behavior and the global structural response. The comparison study with typical modeling approaches in practice confirms that the axial-flexure interaction is critical to capture accurately the deteriorating response and the collapse mechanisms of steel structures.

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\textbf{Introduction}

Over the last decades, seismic hazard has caused severe damage to a wide variety of structural types, ranging from reinforced concrete (RC), masonry, wood, to steel, which is widely believed to exhibit superior performance during earthquakes. Damage in steel structures, even though not as severe and frequent as in other structural types like RC, also results in significant economic loss and thus deserves more attention. Examples of collapsed steel structures include the 6-story Cordova building during the Prince William Sound, Alaska earthquake in 1964, and the 21-story building in the Pino Suarez Complex during the Mexico City earthquake in 1985 \cite{6}. To prevent collapse and provide effective earthquake-resistant designs, it is critical to accomplish the following two aspects simultaneously: (1) to explicitly describe with sufficient accuracy the degrading force-deformation relation in the structural component response, and (2) to quantify the damage states of the components and the structure in a consistent manner with experimental observations.

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Recently, Do and Filippou proposed new beam and column element models based on damage-plasticity to address both aspects in the response simulation and the damage assessment of steel structures [1, 2]. The new models capture accurately the strength and stiffness deterioration in the force-deformation relation as well as quantify consistently the damage evolution in structural components with a continuously updated damage variable. With the ability to simulate the collapse behavior and the damage evolution of structures, the new element models are well suited for collapse evaluation of steel structures under extreme loading condition. This paper adopts the element models in a large-scale building evaluation to showcase the capabilities of the proposed damage formulation to capture the local and the global behavior under high seismic intensity.

**Damage Models for Steel Members**

The element formulation combines resultant-plasticity with damage mechanics to capture the kinematic and isotropic hardening behavior, the cumulative plastic deformations, and the strength and stiffness deterioration in the hysteretic behavior of steel components. The models adopt a criterion based on the hysteretic energy and the maximum or minimum deformation for damage initiation and a cumulative probability distribution function for the damage evolution [1].

**Damage-Plasticity Beam Element**

In the beam model, the axial response is assumed linear elastic and uncoupled from the flexural response. This is a reasonable assumption for girders with negligible axial forces. The inelastic response is monitored at two plastic hinges that are placed inside the element with an offset from the element ends to model the reduced beam sections (RBS). The beam element has been calibrated against experimental measurements of more than 50 steel components under various load patterns [2]. Figure 1 illustrates sample response simulations of the two identical steel specimens under two cyclic load histories [7]. The same parameters are specified in the two simulations according to the guidelines in [2].

![Figure 1. Response simulations of steel beams under different load patterns](image-url)
Damage-Plasticity Column Element

The formulation of the column element resembles the beam element except for the following two modifications. First, the column model utilizes a continuously evolving interaction envelope to describe the inelastic axial behavior and the axial-flexure coupling, which are critical due to considerable axial forces in the column. Second, the strength and stiffness deterioration depends on both the plastic axial energy and the plastic flexural energy to capture the effect of variable axial loads on the deterioration in flexure. The column element has been validated against experimental measurements of steel columns with various sizes, boundary conditions, and load histories [2]. Figure 2 shows sample simulations of two identical steel cantilever columns under different load patterns [3]. The model represents reasonably well the more severe strength deterioration under higher axial compression and the non-symmetric response due to the variable axial load history.

![Figure 2. Response simulation of cantilever column subjected to different axial load patterns: constant axial load (left) and variable axial load (right)](image)

Damage Variables

A unique feature of the damage-plasticity beam and column models is the damage variables $d^+$ and $d^-$ to describe the deterioration in the response under positive and negative moments in each plastic hinge. The continuous updating of the damage variables on the basis of a continuous damage evolution matches the continuous degradation of structural elements under cyclic load reversals in contrast to models with discrete updates at the end of each half cycle [1]. The damage variables are in agreement with the widely-used Park-Ang damage index [5] under common loading scenarios [1, 2]. Response simulations of more than 50 steel component specimens permit a correlation of the damage variables with typical limit states of steel components [2], as summarized in Table 1. The subsequent analyses adopt the threshold $d = 0.6$ for the failure of the structural members.
Table 1. Damage variables and limit states

<table>
<thead>
<tr>
<th>Damage variable</th>
<th>Limit state</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 – 0.2</td>
<td>Initiation of yielding in structural member</td>
</tr>
<tr>
<td>0.2 – 0.4</td>
<td>Initiation of buckling and strength deterioration</td>
</tr>
<tr>
<td>0.4 – 0.6</td>
<td>Significant local buckling and/or cracks propagation</td>
</tr>
<tr>
<td>0.6 – 1.0</td>
<td>Failure from severe local buckling and/or fracture</td>
</tr>
</tbody>
</table>

Case Study: 8-Story 3-Bay Steel Moment-Resisting Frame

To showcase the capabilities of the damage-plasticity beam-column element models, the following sections deploy the models in a case study to investigate the dynamic response of an archetype structure. The archetype structure is an 8-story 3-bay steel special moment frame (SMF) in the PG-2RSA performance group from the NIST evaluation of the FEMA P695 methodology [4]. The columns and girders are wide-flange sections designed for the seismic design category D_{max}. The geometry of the archetype structure is depicted in Figure 3.

![Figure 3. Archetype 8-story 3-bay steel moment-resisting frame](image)

The columns and girders are modeled with the damage-plasticity beam-column elements. The model parameters are calibrated in accordance to the guidelines in [2]. Since the objective of this study is to investigate the deterioration in the structural members, it is assumed that the panel zones induce negligible shear deformations and strength deterioration, and the effect of floor slabs on the element yield strength and flexural stiffness is neglected. Inclusion of these contributions to the global strength and stiffness deterioration is left for future studies.

Dynamic Response

Figure 3a shows the inter-story drift distribution of the archetype structure subjected to the
Hector Mine ground motion scaled by a factor $SF = 6.6$. A large pulse at $t = 7$ sec initiates a 2-story mechanism, as evident in the excessive drifts in the bottom two stories and leads to a one-sided relative motion in the positive direction.

Figure 3b plots the damage distribution of the moment-frame at the end of the ground motion. The plastic hinges are represented with the red circles and the corresponding damage variables are also displayed. For simplicity, the damage variable $d = \max (d^+, d^-)$ is used to represent the damage in the plastic hinges. The damage distribution is highly non-symmetric with higher damage on the right side of the structure. Damage concentrates at the base of the first-story columns, at the top of the second-story columns, and in the first-floor girders. This damage distribution is consistent with the two-story collapse mechanism observed in the time history analysis. Let C-1-1 and C-1-4 denote the exterior columns on the left and right in the first-story, respectively. The high damage value at the base of C-1-4 indicates severe local buckling. A non-symmetric damage distribution results from the considerable discrepancies in the plastic energy dissipation in the columns. Such discrepancies are typically due to the larger drift in one direction that amplifies the overturning effect and induces more plastic axial deformations in columns on one side than the other. In a special case, the structure initiates a weak-story mechanism and permanent deformations in one dominant direction, which exacerbates the discrepancies in the column plastic axial deformations, and in turns, leads to a non-symmetric damage distribution. It is noteworthy that the damage in the interior columns is relatively mild because of the higher ductility of the larger section size. Moreover, damage in the interior columns is more symmetric due to the lower impact from the overturning effect.

![Figure 3: Story drift distribution (left) and damage distribution (right)](image)

Figure 4 compares the response at the base of C-1-1 on the left and C-1-4 on the right. The comparison of the moment-rotation relation in Figure 4a gives rise to two observations: (1) the columns accumulate similar flexural deformations, and (2) the strength and stiffness deterioration is more pronounced in C-1-4 than in C-1-1. The first observation suggests that the axial deformations govern the discrepancies in the column response, and the flexural response only is inadequate to assess the column behavior. Due to the relative motion of the structure causing permanent deformations in the positive direction, the overturning effect induces higher compression and more excessive plastic axial deformations on C-1-4, as Figure 4b shows. The
Evolution of the plastic axial deformations in Figure 4b correlates well with the damage evolution in Figure 4c. After the large pulse at $t = 7$ sec triggers the weak-story mechanism, the accumulation of plastic axial deformations in C-1-4 accelerates due to the overturning effect and leads to higher damage increments than in C-1-1.

![Figure 4: Response of first-story exterior columns: (a) base moment, (b) plastic axial deformations, (c) damage evolution](image)

**Effect of Axial Load on Deterioration in Flexure**

The damage-plasticity element models account for two unique features that capture complex global and local behavior up to collapse: (1) the strength and stiffness deterioration in the element response, and (2) the inelastic axial force-deformation relation and the effect of variable axial forces on the flexural behavior. In practice, one or both aspects are often neglected in the simulations due to limitations of available element options. In this section, the structural model in the preceding sections is used as a reference (model R) to compare against an alternative that represents a commonly-used modeling approach:

- Model R: the reference model, in which the strength and stiffness deterioration of the girders and columns are accounted for in the damage-plasticity beam-column elements.
- Model B: same as Model R, but the column response does not account for the inelastic axial behavior and the axial-flexure interaction. The column element in model B does not utilize an interaction envelope to account for the axial-flexure coupling.

The dynamic response of the archetype moment-frame under the Superstition Hills ground motion at the Poe Road station scaled by $S_F = 9.4$ highlights the key differences in model R and model B. Figure 5 plots the evolution and the distribution of story drifts. It is evident in the first-story drift evolution in Figure 5a that the structure in model B remains stable while model R indicates loss of stability. The residual deformations in model B are relative small and do not initiate a weak-story mechanism whereas the structure in model R collapses in a 2-story mechanism with considerable drifts in the bottom 2 stories.
Figure 5: Time history of first-story drift (left) and final inter-story drift distribution (right)

Figure 6 compares the response of left exterior column in the 1st story (C-1-1) between the two modeling approaches. Model B underestimates the element damage and fails to capture the excessive strength and stiffness deterioration in the column hysteretic behavior as well as the drastic increment in the flexural deformation near the end of the ground motion, as evident in the flexural response shown in Figure 6a. As expected, the two models differ tremendously in the axial response. As Figure 6b shows, while model B gives a linear elastic response with limited axial deformation, model R captures the plastic axial deformations accumulated as the column yields. The differences in the plastic deformations are reflected in the evolution of the damage variables at the column base in Figure 6c. Without accounting for the plastic axial deformations and the effect of the axial response on the deterioration in flexure, model B is unable to simulate the severe damage in the column.

Figure 6: Response of exterior first-story column on the left: (a) flexural response, (b) axial response, (c) damage evolution at column base

The damage distribution also highlights the significance of the axial-flexure interaction in the deterioration modeling of structures. Figure 7 compares the damage distribution in the moment-frame at the end of the ground motion between the two modeling approaches. The damage profiles are consistent with the global and the local response. While model R indicates a two-story mechanism with high damage concentration in the members in the bottom two stories, model B suggests a different mechanism with more damage localized in the girders. The symmetry of the damage distribution also distinguishes the two models and emphasizes the
effect of axial force on element damage. In model B, without the inelastic axial behavior, damage in the columns depends solely on the flexural response. On the same floor, the elements accumulate the same plastic flexural deformations, and thus, leading to a symmetric damage profile. The inelastic axial response and the axial-flexure interaction allow model R to capture the non-symmetric damage distribution due to the considerable overturning effect.

Concluding Remarks

This paper adopts a novel approach to model the strength and stiffness deterioration in the response of column and girder members of steel structures. The damage-plasticity beam and column elements describe the degrading element response on the basis of a damage variable that is continuously updated based on the hysteretic energy dissipation and the maximum or minimum deformation. The elements are deployed in a pilot study of an 8-story 3-bay steel special moment-resisting frame that is subjected to far-field ground motions with increasing intensity up to collapse. The model captures sufficiently the key characteristics of steel components, for instance, the accumulation of plastic deformations, the continuous strength and stiffness deterioration, and in particular, the strength reduction due to variable axial loads in the columns. The distribution of the damage variables within the structure identifies the severely degrading members and highlights the impact of the overturning effect on the column strength deterioration in the lower stories. The damage distribution is indicative of both the local element response and the global structural behavior, and describes consistently the weak story collapse mechanism observed in the time history analyses. The comparison study of the two modeling approaches highlights the significance of the effect of the axial response on the deterioration in flexure. Without the axial-flexure interaction, the structural model overestimates the column strength and, in turns, underestimates the severe deterioration in the local and global response. With these unique features, the new damage-plasticity elements are well suited for large-scale simulations and collapse assessment of steel structures under extreme loading conditions.
References


