Toward seismic design of steel frames using an energy-based damage index

E. Bojórquez & A. Reyes-Salazar
Facultad de Ingeniería, Universidad Autónoma de Sinaloa, México

A. Terán-Gilmore
Departamento de Materiales, Universidad Autónoma Metropolitana, México

S.E. Ruiz
Instituto de Ingeniería, Universidad Nacional Autónoma de México, México

ABSTRACT:

A new energy-based damage model that accounts explicitly for the effects of cumulative plastic deformation demands is introduced with the purpose of assessing the seismic performance of multi-degree-of-freedom (MDOF) steel structures. The model was developed from the results obtained during experimental testing of steel members, and from analytical studies regarding the height distribution of plastic demands on several moment-resisting steel frames designed according to the Mexico City Building Code. The pertinence of using simplified systems to evaluate structural damage in MDOF steel structures is discussed. The results suggest that structural damage can be evaluated in a reasonable manner through the use of simple models; and thus, that damage evaluation of MDOF systems can be accomplished through small to moderate analytical effort. Finally, it is concluded that a simple energy-based evaluation methodology can be implemented for seismic design purposes of moment-resisting steel frames through the use of normalized hysteretic energy spectra.

Keywords: energy-based damage index, steel structures, cumulative plastic deformation demands

1. INTRODUCTION

Several researchers have recently discussed and shown the importance of ground motion duration in the structural performance of buildings due to the effect of cumulative plastic deformation demands. Because of this, several studies have been devoted to calibrate damage indexes for steel and reinforced concrete members with the aim of better representing the consequences of earthquake duration (Krawinkler and Zohrei 1983, Park and Ang 1985, Bozorgnia and Bertero 2001, Terán and Jirsa 2005, Rodriguez and Padilla 2008); however, currently there is a challenge to study and calibrate the use of such indexes for the practical structural evaluation of complex structures. Particularly, it is necessary to establish appropriate parameters for an adequate numerical evaluation during the performance based-design of structures. The effect of cumulative plastic deformation demands can be considered through the use of energy concepts; especially through the plastic dissipated hysteretic energy demand. The use of energy for this purpose was initially discussed by Housner (1956), and has been used by several researchers to propose energy-based methodologies that aim at providing earthquake-resistant structures with an energy dissipating capacity larger or equal than its corresponding demand (Akiyama 1985, Akbas et al. 2001, Choi and Kim 2006, Bojórquez et al. 2008). In the present study, a new energy-based damage model for MDOF steel framed structures is introduced. The model was developed from information derived from experimental testing of steel members. Firstly, it will be demonstrated that the uncertainties in the energy dissipation capacity of the steel structural members of a frame are not significant to the use of the new energy-based damage model for evaluation purposes. Moreover, the use of simplified systems to evaluate structural damage in MDOF steel structures is analyzed. Finally, it is concluded that a simple energy-based evaluation procedure can be developed for the seismic design of moment-resisting steel frames through the use of normalized hysteretic energy spectra.
2. ENERGY-BASED STRUCTURAL DAMAGE MODEL

Energy-based methodologies are focused at providing structures with energy dissipating capacities that are larger or equal than their expected energy demands (Akiyama 1985, Uang and Bertero 1990). The design requirement of an earthquake-resistant structure in these terms can be formulated as:

$$\text{Energy Capacity} \geq \text{Energy Demand}$$ \hspace{1cm} (2.1)

Among all the energies absorbed and dissipated by a structure, the plastic dissipated hysteretic energy $E_{HC}$ is clearly related to structural damage. $E_{HC}$ can be physically interpreted by considering that it is equal to the total area under all the hysteresis loops that a structure undergoes during a ground motion. Therefore, it is convenient to express Eqn. 2.1 in terms of plastic dissipated hysteretic energy:

$$E_{HC} \geq E_{HD}$$ \hspace{1cm} (2.2)

where $E_{HC}$ is the plastic hysteretic energy capacity and $E_{HD}$ is its corresponding energy demand. Eqn. 2.2 can be reformulated as an energy-based damage index as follows:

$$I_{DE} = \frac{E_{HD}}{E_{HC}} \leq 1$$ \hspace{1cm} (2.3)

In Eqn. 2.3, the performance level or condition characterized by $E_{HD} = E_{HC}$, is considered to correspond to failure of the system. Hence, while $I_{DE} = 1$ corresponds to failure of the structural system; a value of zero implies no structural damage (elastic behavior implies no structural damage). From a physical point of view, this equation represents a balance between the structural capacity and demand in terms of energy. In this sense, this formulation follows the direction initially established by Housner in 1956 for an energy-based design.

According to Eqn. 2.3, structural damage depends on the balance between the plastic hysteretic energy capacity and demand on the structure. While the plastic hysteretic energy demand can be obtained through dynamic analysis, a challenge exists to define the plastic hysteretic energy capacity of a structure. Nevertheless, flexural plastic behavior is usually concentrated at the ends of the structural members that make up a frame; and in the particular case of W steel shapes, in the flanges. In these terms, the plastic hysteretic energy capacity of a steel member that forms part of a structural frame can be estimated as follows (Akbas et al., 2001):

$$E_{HCm} = 2Z_f f_y \theta_{pa}$$ \hspace{1cm} (2.4)

where $Z_f$ is the section modulus of the flanges of the steel member; $f_y$, the yield stress; and $\theta_{pa}$, its cumulative plastic rotation capacity. Note that the above equation considers that plastic energy is dissipated exclusively through plastic behavior at both ends of a steel member.

Eqn. 2.4 can be used together with Eqn. 2.3 to evaluate the level of structural damage in steel members. However, it is convenient to normalize the hysteretic energy ($E_h$) for damage evaluation purposes (Krawinkler and Nassar 1992, Terán-Gilmore and Simon 2006):

$$E_h = \frac{E_{H}}{F_y \delta_y}$$ \hspace{1cm} (2.5)
where \( F_y \) and \( \delta_y \) are the strength and displacement at first yield, respectively. Eqn. 2.3 can be re-expressed in terms of \( E_N \) as follows:

\[
I_{DEN} = \frac{E_{ND}}{E_{NC}} \leq 1
\]

(2.6)

where the parameters involved in Eqn. 2.6 have the same meaning as those used in Eqn. 2.3. The advantage of formulating the problem in terms of \( E_N \) is that this is a more stable parameter, and in quantitative terms it can easily be used for practical purposes. In other words, the energy-based damage index proposed herein corresponds to the ratio between the normalized hysteretic energy demand and normalized hysteretic energy capacity, and the condition of failure is assumed to be \( I_{DEN} \) equal to one.

In the case of MDOF steel structures, the principal challenge for the practical use of Eq. 2.6 is the definition of the energy capacity of the structure in terms of that of its structural members. Through the consideration that in regular steel frames the energy is dissipated exclusively by the beams (which is an appropriate hypothesis for strong column-weak beam structural systems), the energy capacity of these systems can be estimated as (Bojórquez et al., 2008):

\[
E_{NC} = \sum_{i=1}^{N_S} \left( 2 \cdot N_B \cdot Z_j \cdot F \cdot \theta_{pa} \cdot F_{EH} \right) \frac{C_y \cdot D_y \cdot W}{C_y \cdot D_y \cdot W}
\]

(2.7)

where \( N_S \) and \( N_B \) are the number of stories and bays in the frame, respectively; \( F_{EH} \), an energy participation factor that accounts for the different contribution of each story to the energy dissipation capacity of the frame; \( W \), the total weight of the frame; and finally, \( C_y \) and \( D_y \), the seismic coefficient and displacement at first yield, respectively, estimated for the frame from a static nonlinear analysis.

Eqn. 2.7 shows the role of the cumulative plastic rotation capacity of the structural members in the total energy dissipation capacity of a frame. A wide range of \( \theta_{pa} \) values was collected by Akbas (1997) from experimental tests of steel members subjected to cyclic loading. Based on these values, Bojórquez et al. (2009) found that the cumulative plastic rotation capacity of steel members is well represented by a lognormal probability density function with a median value equal to 0.23.

### 3. IMPACT OF UNCERTAINTY IN CUMULATIVE PLASTIC ROTATION

The impact of the uncertainty in the rotation capacity of the structural members on the damage evaluation of a frame is analyzed next. With this aim, six regular steel frames designed according to the Mexico City Building Code were subjected to 23 soft-soil long duration ground motions recorded in the Lake Zone of Mexico City and exhibiting a dominant period \( (T_s) \) of two seconds. Particularly, all motions were recorded during seismic events with magnitudes of seven or larger, and having epicenters located at distances of 300 km or more from Mexico City. The frames, which were assumed to be used for office occupancy, have three bays and a number of levels that range from four to eighteen. The bay and inter-story dimensions are those indicated in Fig. 3.1a. The frames were designed for ductile detailing. A36 steel and W sections were used for the beams and columns of the frames. An elasto-plastic model with 3% strain-hardening was considered to model the cyclic behavior of the steel members (Bojórquez and Rivera 2008). The critical damping ratio was assumed equal to 3%. Relevant characteristics for each frame, such as the fundamental period of vibration \( (T_1) \), and the seismic coefficient and displacement at first yield \( (C_y \) and \( D_y \)) are shown in Table 1 (the latter two values were established from static nonlinear analyses). Note that the frames exhibit a wide range of period values. Fig. 3.1b shows the response spectra of the seismic records under consideration. To study the influence
of the uncertainty in the cumulative plastic rotation capacity on the evaluation of structural damage through the use of Eqn. 2.6, a lognormal probability density function with a median value of 0.23 was used to describe the variation of the cumulative plastic rotation capacity at the ends of the beams (Bojórquez et al. 2009). For illustration purposes, four standard deviations of the natural logarithm were considered: 0, 0.1, 0.3 and 0.5. Note that a standard deviation of zero corresponds to mean values. While it was assumed that the value of $\theta_{pa}$ varied through height according to the lognormal density function, the value of $\theta_{pa}$ for all beams within a story was considered the same.

![Figure 3.1.](image)

**Figure 3.1.** a) Geometrical characteristics of steel frames; b) Elastic response spectra of the selected ground motion records

<table>
<thead>
<tr>
<th>Frame</th>
<th>Number of Stories</th>
<th>$T_1$ (s)</th>
<th>$C_y$ (m)</th>
<th>$D_y$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F4</td>
<td>4</td>
<td>0.90</td>
<td>0.45</td>
<td>0.136</td>
</tr>
<tr>
<td>F6</td>
<td>6</td>
<td>1.07</td>
<td>0.42</td>
<td>0.174</td>
</tr>
<tr>
<td>F8</td>
<td>8</td>
<td>1.20</td>
<td>0.38</td>
<td>0.192</td>
</tr>
<tr>
<td>F10</td>
<td>10</td>
<td>1.37</td>
<td>0.36</td>
<td>0.226</td>
</tr>
<tr>
<td>F14</td>
<td>14</td>
<td>1.91</td>
<td>0.25</td>
<td>0.30</td>
</tr>
<tr>
<td>F18</td>
<td>18</td>
<td>2.53</td>
<td>0.185</td>
<td>0.41</td>
</tr>
</tbody>
</table>

The influence of the uncertainty in the cumulative plastic rotation capacity is illustrated through the results obtained from incremental dynamic analysis of all frames under consideration. For this purpose, the frames were subjected to the selected ground motions, scaled up in such manner as to achieve the same spectral ordinate at the period corresponding to the first mode of vibration of each particular frame. A wide range of motion intensities were considered for this purpose. Fig. 3.2 shows and compares the median values of $I_{DEN}$ obtained from Eq. 2.6 for all the steel frames under consideration. The horizontal axis considers the different intensity levels quantified through the spectral acceleration associated to the first mode of vibration. The comparison suggests that there is no significant influence of the level of uncertainty of $\theta_{pa}$ in the damage estimates for all the frames and intensity levels under consideration.

In general, the damage estimated for different levels of uncertainty is quite similar for each particular frame. It can be concluded that reasonable estimates of structural damage can be obtained through the consideration of median cumulative rotation capacities.
Figure 3.2. Structural damage estimated from incremental dynamic analysis of the frames under the consideration of different levels of uncertainty in the cumulative plastic rotation capacity.

4. EVALUATION OF STRUCTURAL DAMAGE WITH SIMPLIFIED SYSTEMS

Although the estimation of overall structural damage in steel frames can be carried out, as discussed previously, through a relatively pertinent and simple formulation, full nonlinear dynamic analyses need to be carried out to estimate the plastic energy demands. Within this context, it is convenient to simplify the estimation of the plastic energy demands through reducing the MDOF structure to an equivalent single-degree-of-freedom (SDOF) system. This can be done in turn through the consideration of a plastic hysteretic energy transformation factor (Bojórquez and Ruiz 2007). If such transformation factor is available, the structural damage in a MDOF regular steel frame subjected to long duration ground motion records could be estimated from the results derived from the nonlinear dynamic analysis of an equivalent SDOF system, or from normalized plastic hysteretic energy spectra. Within this context, the first step to evaluate damage in a MDOF structure subjected to seismic loading is to establish its
equivalent SDOF system. Particularly, it is proposed herein that the equivalent SDOF system needs to have the same structural period, seismic coefficient and critical damping as those exhibited by the MDOF structure.

Bojórquez and Ruiz (2007) established that the normalized plastic hysteretic energy transformation factor \((T_{EN})\) is close to 3 for regular steel buildings subjected to long duration ground motions. Once a nonlinear dynamic analysis is performed to evaluate the normalized plastic hysteretic energy demand in the equivalent SDOF system, the normalized plastic hysteretic energy in the MDOF structure can be estimated as follows:

\[
E_{ND}(MDOF) = \frac{E_{ND}(SDOF)}{T_{EN}} \tag{4.1}
\]

By substituting Eqns. 2.7 and 4.1 into Eqn. 2.6, structural damage can be evaluated in a MDOF structures through the analysis of SDOF systems.

To discuss herein the pertinence of using the simplified procedure formulated in terms of an equivalent SDOF system, the level of structural damage in the frames was evaluated directly from their MDOF models, and compared to that estimated using the simplified approach. For this purpose, incremental dynamic analyses of the frames were carried out, and their level of structural damage evaluated using Eqn. 2.6. The frames and their equivalent SDOF systems were subjected to the ground motions under consideration, which were scaled up in such manner as to achieve the same spectral ordinate for all motions when evaluated at the period corresponding to the first mode of vibration of each frame. A wide range of motion intensity was considered for this purpose. Fig. 4.1 shows and compares the median value of \(I_{DEN}\) obtained from Eqn. 2.6 for the MDOF models of frames F10 and F14, and for its corresponding equivalent SDOF systems. The horizontal axis considers the different intensity levels given to the spectral acceleration associated to the first mode of vibration, \(S_a(T_1)\). The comparison established through Fig. 4.1 suggests that structural damage is estimated reasonably well through the use of a SDOF system. This means the equivalent SDOF approach is not only practical, but also useful for evaluating the structural performance of MDOF steel frames. In particular, for all the intensity levels under consideration, damage is estimated similarly with the MDOF and SDOF models, in such a way that the approach discussed herein seems promising for practical application.

\[\text{Figure 4.1. Comparison of median values of structural damage for frames F10 and F14, MDOF versus SDOF approach}\]
5. TOWARD SEISMIC DESIGN OF STEEL FRAMES STRUCTURES USING AN ENERGY-BASED DAMAGE INDEX

Because it provides a simple manner to estimate the maximum seismic demands on earthquake-resistant structures, one of the most important tools for seismic design is a well formulated seismic design spectrum. Nevertheless, several issues should be taken into consideration when formulating appropriate design spectra within a code format. A first issue that will be discussed herein is the inclusion of the effect of cumulative deformation demands during the seismic design of an earthquake-resistant structure. In this paper, cumulative demands are explicitly contemplated through the formulation of a damage index that makes explicit consideration of the cumulative plastic deformation demands. It was observed that the use of this energy-based damage index within a design context formulated in terms of a SDOF model results in reasonable estimates of structural damage in regular steel frames. Within this context, energy spectra can be used for structural evaluation of complex structures, and thus constitute a relevant tool for structural assessment. Note that the energy-based evaluation procedure was formulated in terms of the energy dissipation capacity of earthquake-resistant structures, in such way that once this capacity is established by using information provided from experimental testing of steel elements, the only other information required in the form of an energy demand can be derived from normalized hysteretic energy spectra.

A second issue that will be addressed is the inclusion of specific reliability levels associated to the structures to be designed. Regarding this, Bojórquez et al. (2008) proposed an earthquake-resistant evaluation procedure for steel frames that considers the use of normalized hysteretic energy spectra with uniform annual failure rates. The combination of the energy-based evaluation procedure developed in this paper with the formulation of energy spectra with uniform annual failure rates constitute itself on an excellent alternative to incorporate cumulative demands within specific structural reliability settings.

6. CONCLUSIONS

An energy-based damage model for multi-degree-of-freedom steel structures has been proposed. The model is based on the demand-supply balance of plastic hysteretic energy. Particularly, the damage index is formulated as the ratio of the normalized plastic hysteretic energy demand to its corresponding capacity. While a value of zero for the damage model implies no structural damage, a unitary value implies failure. No significant influence of the uncertainty in the cumulative plastic rotation capacity of the structural elements was observed in the structural evaluation of regular steel frames. Moreover, a simplified approach for the evaluation of structural damage was introduced. For this purpose, an equivalent SDOF system was formulated. The results presented herein, obtained from nonlinear dynamic analyses of MDOF steel frames, indicate that structural damage can be adequately estimated through the use of a SDOF approach. The energy-based damage model introduced herein is a promising tool for the evaluation of the seismic performance of structures subjected to long duration ground motion. In these terms, the tool can be used for the formulation of code related design requirements for steel frames that may be subjected to severe cumulative plastic deformation demands.

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REFERENCES


Bojórquez, E. and Ruiz S.E. (2007). Factores de transformación de ductilidades, distorsiones máximas de entrepiso y de energía histerética normalizada entre S1GL y SMGL. Third National Conference on Earthquake Engineering, Girona, Spain. (in Spanish)


