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SEISMIC DAMAGE MITIGATION AND EVALUATION OF STEEL MOMENT RESISTING FRAMES

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ABSTRACT

Two important issues regarding steel moment resisting frames (MRFs) are addressed in this research. First, an improved design of Welded–Flanges–Bolted–Web (WFBW) moment resisting steel connections was investigated to mitigate the brittle damage observed during some recent earthquakes. Analytical investigations were conducted on models of individual modified connections as well as MRFs with the modified moment resisting connections using *DRAIN–2DX*. The analytical results indicated that the proposed modification to the beam ends could be an effective means to mitigate brittle connection damage that has occurred in WFBW connections during recent seismic events without inducing any unfavorable effects on the overall seismic behavior of the MRF. The second part of the research was the development of a systematic seismic damage assessment approach for steel MRFs. A low–cycle fatigue connection damage model was developed, which related the damage in a connection directly to the hysteretic energy dissipated in the connection. Based on this connection damage model and a weight–averaging technique, a systematic damage assessment approach for steel MRFs was studied. The overall damage assessment approach made it possible to assess quantitatively damage to steel MRFs. This damage assessment technique will provide structural engineers with another valuable tool for the design, retrofit and post–earthquake damage evaluation of steel frame structures.

KEYWORDS

earthquake engineering; steel frame; seismic damage; damage model; damage assessment; damage mitigation; moment resisting connection; moment resisting frame; hysteretic energy; low–cycle fatigue.

INTRODUCTION

Unprecedented cases of brittle and unpredicted failure of Welded–Flanges–Bolted–Web (WFBW) connections were found in steel moment resisting frames during some recent earthquakes. This type of connection has been widely used in buildings with steel frame construction in seismic areas. Therefore, developing methods to retrofit existing connections and design new connections became an urgent need within the steel design community in the United States and elsewhere. In most cases, the brittle failures began with the initiation of cracks in the welds at the beam–column interface after the welds were overstressed by transverse loads. Consequently, if the stresses developed at the beam–column interface during a large earthquake ground motion can be reduced, the brittle behavior of such connections could be accordingly reduced or even eliminated. In the first part of this research, a method of modifying moment resisting connections to enhance the connection ductility was proposed. It was based on the concept that a beam section weak in bending near the beam–column interface could be created to reduce the stress developed at the interface during a cyclic loading process.

The issue of quantifying structural steel damage resulting from an earthquake is of great importance. Being able to quantitatively assess the seismic damage of a steel connection is the first step towards the development of a rational seismic damage assessment approach for steel structures. The achievements of previous research on damage analysis of steel structures were mainly in the low–cycle fatigue damage behavior of cyclically loaded simple structural components. A connection is not a simple structural component since it is composed of beams, columns, welds, etc.. Research has been performed to study cyclic damage characteristics of moment connections (Krawinkler and Zohrei, 1983, Castiglioni and Losa, 1992). It was concluded that the connections under cyclic loading exhibited low–cycle fatigue damage behavior. In the second phase of this research, a hysteretic energy based connection damage model was developed to quantitatively assess the damage to moment connections. Based on this connection damage model, a systematic damage assessment technique was investigated for steel structures by accumulating the damage to the individual connections to arrive at story level damage and total frame damage.

MODIFIED MOMENT RESISTING CONNECTIONS

During inelastic deformations, when a cross section of the beam near the beam–column interface in a connection is made weak in bending, the first plastic hinge would form at that location. Generally, this beam section is more ductile than the beam section at the beam–column interface where the welds are located. Therefore, during inelastic excursions from an earthquake, the inelastic deformation (energy) can be diverted away from the susceptible beam–column interface to the weakened section. As a result, the ductility of this connections could be increased. A modified moment resisting connection is illustrated in Fig. 1, where d is the depth of the beam section.

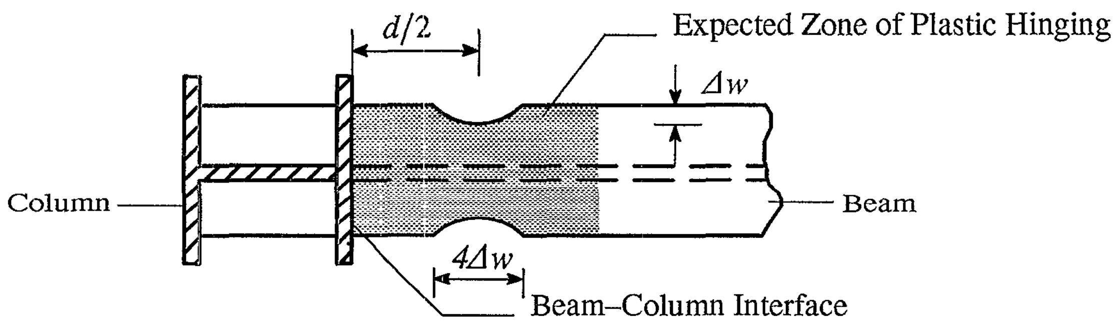

Fig. 1. Modified Moment Resisting Connection

Performance of the Modified Connections

The effectiveness and feasibility of the proposed modified connection was investigated analytically using the structural analysis package *DRAIN–2DX*. The investigation is focused on two aspects: the cyclic behavior of the modified moment resisting connections and the overall seismic behavior of a MRF with modified connections.

Cyclic Behavior of Modified Moment Resisting Connections. The numerical modelling of the connections was calibrated against the full–scale cyclic testing data of a moment resisting connection conducted at the University of Texas at Austin (Engelhardt and Husain, 1992). The subassemblage of one of the connections tested at UT is shown in Fig. 2. The same connection subassemblage was used in the analytical study of this research. In the modelling, the columns were modelled using the beam–column element and the beams were modelled using the fiber element.

To study the effectiveness of the improved ductility resulting from the modification, the cyclic behavior of a original connection and four modified connections with different beam moment capacity reductions were studied. The original moment resisting connection investigated was the analytical replica of the connection shown in Fig.2. Other four connections investigated were modified from this original connection. In the first

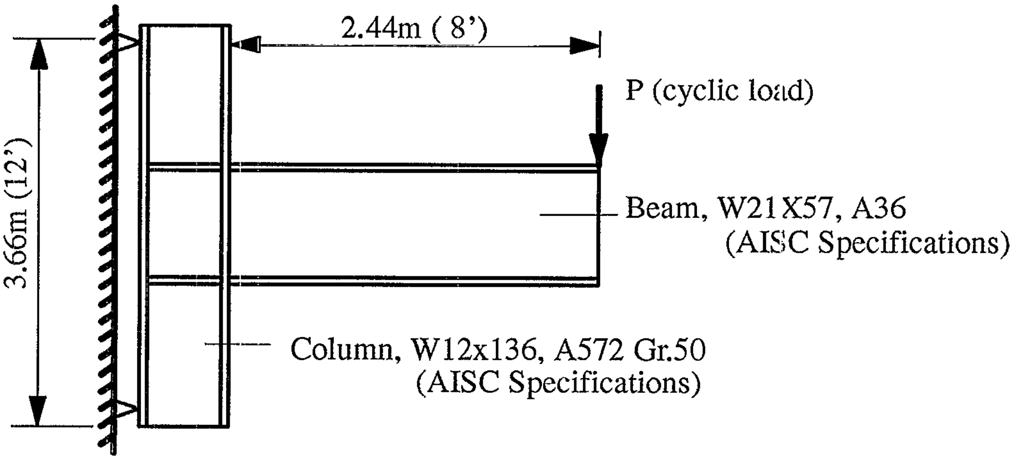

Fig. 2. Connection Subassemblage

modified connection, the moment capacity after the flange area reduction at the location of the weak section was equal to the moment capacity required to resist the moment caused by the lateral load at this location, M_{p1}, which was

$$M_{p1} = \frac{(L - d/2)}{L} M_p \tag{1}$$

where L is the length of the cantilever of the connection, d is the depth of the beam section, and M_p is the plastic moment capacity of the original beam section. In three other cases, the beam capacities at the weakened sections were 90%, 80% and 70% of M_{p1}, respectively. The lateral stiffness reductions resulting from the area reductions of the beam sections were calculated. The strain level reduction at the beam–column interface as a measure of the effectiveness of the modification for each case was obtained through the analysis for a cyclic load. The extreme fiber strains developed at the beam–column interface for different connections during the cyclic load were plotted in Fig. 3.

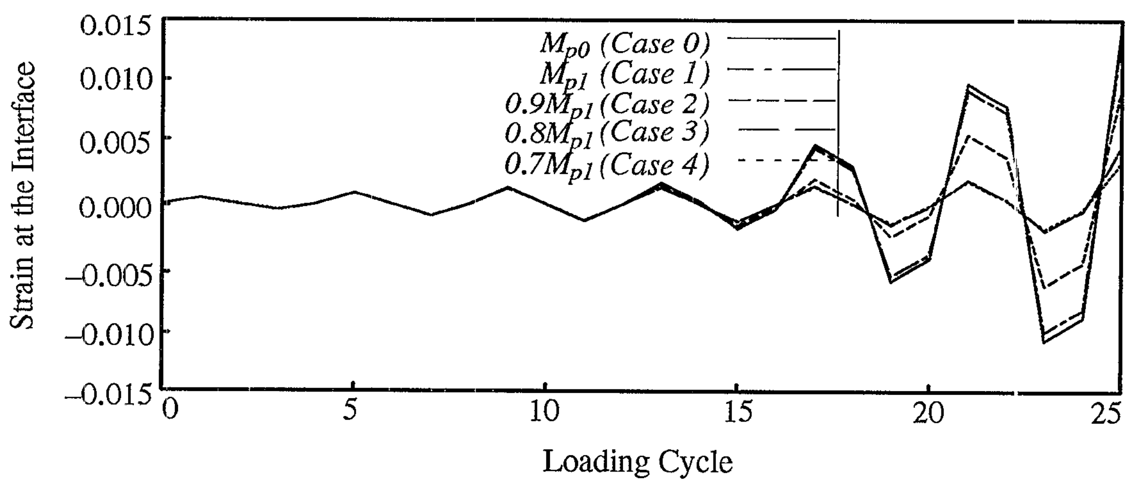

Fig. 3. Extreme Fiber Strains at Beam–Column Interface for a Cyclic Load

For *Case 1*, where the plastic bending capacity at the weak section was M_{p1}, it was found that the strain level at the beam–column interface was reduced only 2.19%. But when the flange area was further reduced as in *Cases 2, 3* and *4*, which means the plastic hinge formed first at the weak section, the strain level at the beam–column interface was reduced to a large amount. In the *Case 2*, with 90% of M_{p1} at the location of the weak section, 20% reduction in the strain level at the interface for a typical inelastic loading cycle was obtained. In *Case 4*, the material at the beam–column interface exhibited almost perfect elastic behavior. The reduction in the transverse stiffness of the moment resisting connections was found to be very small, with the maximum reduction of only 3% in *Case 4*.

Frame with Modified Moment Resisting Connections. One concern about modifying the connections in a MRF was whether the modifications in the beams would unfavorably affect the overall seismic performance of the MRF. Analytical investigations were performed for a MRF with original connections as well as a MRF with modified connections (MMRF). The original MRF was designed in accordance with the current version of the seismic provisions in the *Uniform Building Code*. The MMRF was developed by modifying both ends of the girders in the original MRF. The moment capacity reduction in the weak section of a girder, which was located at a distance *d/2* from the beam–column interface, was chosen to be 90% M_{p1} of the girder. The MMRF is shown in Fig. 4.

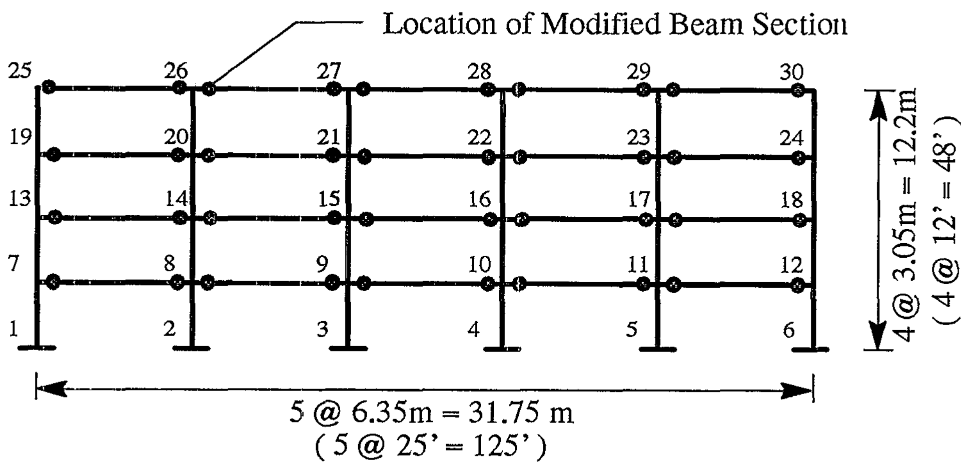

Fig. 4. Modified Moment Resisting Frame

Six California earthquake accelerograms, including a Northridge Earthquake accelerogram, were selected and used in the time–history analyses of the original MRF and the MMRF. Similar to the numerical modelling of the connection described earlier, the columns were modelled using the beam–column element and the beams were modelled using the fiber element. Results from the inelastic time–history analyses for both frames were compared with each other in the following aspects: the maximum story shears, the maximum story drifts and the extreme fiber strain reductions at the beam–column interfaces. The extreme fiber strains at both the interface and the weak section of joint 7 resulted from the Northridge Earthquake are presented in Fig. 5.

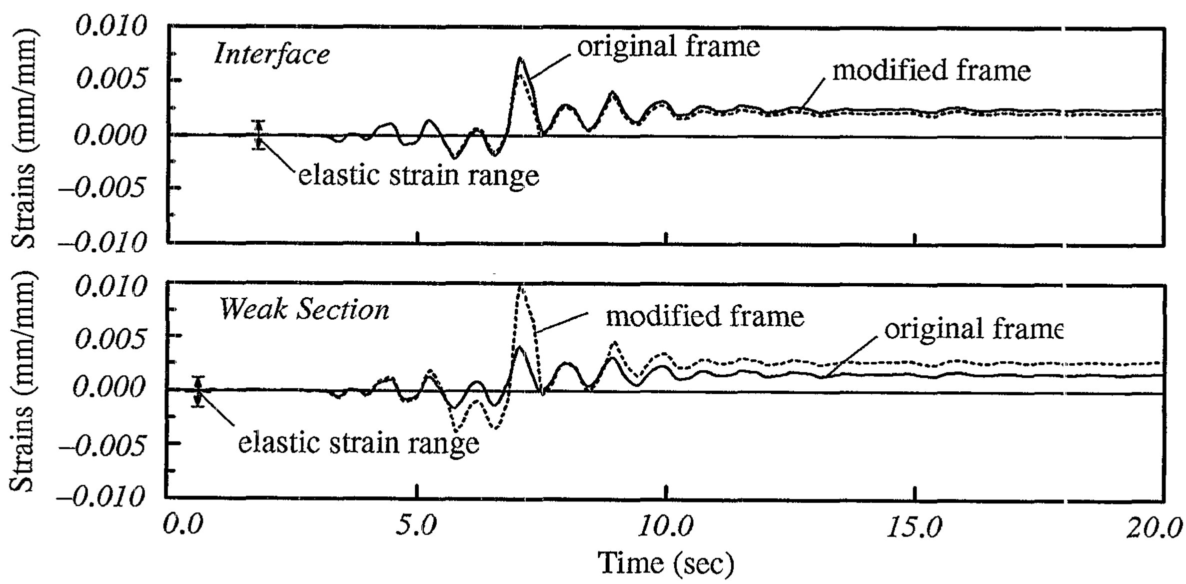

Fig. 5. Extreme Fiber Strains of Joint 7 during Northridge Earthquake

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Because of the different characteristics of the earthquakes, the resulting maximum story shears and story drifts were quite different for the six earthquakes. Generally, it was found that the modifications resulted in slightly smaller maximum story shears (maximum 3% reduction) as well as maximum story drifts (maximum 5% reduction) in the modified frame. It was observed that the achieved reductions of the extreme fiber strains at the beam–column interface in the MMRF were approximate 20% for large inelastic excursions and 5% to 10% for the excursions with small inelastic deformations. Large increases in strains were observed at the weak section. It indicated that the plastic hinge, if there was any, would form first at the location of the weakened section and would dissipate a large amount of hysteretic energy during inelastic deformations.

DAMAGE ASSESSMENT

The overall performance of a frame or building designed to resist a earthquake depends on the seismic performance of the structural components, such as the connections. Therefore, in order to assess the seismic damage of a whole frame, a hysteretic energy based steel connection damage model was developed first. Then, based on this damage model, a systematic damage assessment approach for steel moment resisting frames was investigated.

Hysteretic Energy Based Connection Damage Model

Because of the fact that the damage of a steel component in a structure subjected to a seismic ground motion is in essence a low–cycle fatigue damage process, the low–cycle fatigue theory has been applied in the seismic damage analysis of a steel component. Past experiments have demonstrated that the damage of a cyclically loaded steel connection exhibited low–cycle fatigue behavior, although the damage characteristics depended not only on steel material properties, but also on other factors such as, connection geometry, sectional properties, the welding quality. In this research, the relationship between the damage occurring in a moment resisting connection with a beam of W section and the hysteretic energy dissipated in the connection was established based on three major assumptions: 1) Manson's universal slope, 2) bilinear stress–strain relationship of the steel material and bilinear moment–rotation relationship of the moment resisting connection, and 3) linear damage accumulation rule. The level of damage occurring in a connection was described through the use of a damage index, which ranges from zero (no damage) to one (total damage):

$$DI = C_0 \sum_{i=1}^{N} \Delta E_i^{\frac{1}{0.6}} \qquad (2)$$

where ΔE_i is the amount of hysteretic energy dissipated in the connection during a loading reversal i, which can be obtained from an inelastic time–history analysis, and C_0 is a parameter which depends on the geometry of the connection, sectional properties of the beam in the connection, material properties and the fatigue ductility coefficient of the connection, ε_f'. The fatigue ductility coefficient for a connection depended on factors such as, steel properties, member sizes, weld quality of the connection. Each type of connection possesses a specific value for the fatigue ductility coefficient which could be obtained from experiments. This connection damage model was evaluated by examining the values of the ductility fatigue coefficients calculated from the data of full–scale connection tests. Because of the fact that the calculated values did reflect the physical meaning of the fatigue ductility coefficient which was recognized by the pioneers in the metal fatigue field, this model was judged to be an appropriate connection damage evaluation model. The value of the fatigue ductility coefficient reflected the ductility of the connection and, similar to other material properties, followed a random distribution. Based on data of three sets of full–scale tests on WFBW connections of typical sizes (Tsai and Popov, 1988, Tsai, *et al.*, 1995 and Engelhardt and Husain, 1992), the most probable value of the fatigue ductility coefficient for WFBW type connections was statistically found.

Global Damage Assessment

The approach to assess the level of damage of a moment resisting frame could follow a systematic procedure, in which damage is assessed from the structural components level, to the level of each story, and finally to the overall frame. As proposed in this research, the damage index for a joint was found through the damage information of the local structural components, the damage index for a story was then calculated form the joint damage indices, and eventually, the damage index of the overall moment resisting frame was obtained based on damage information of the stories.

Joint Damage Index. The damage in a steel moment resisting frame caused by an earthquake ground motion is expected to occur in the beam–column joint zone, or at the ends of columns or beams framing into the joint. Theoretically, steel MRFs designed according to the Strong–Column–Weak–Beam requirements with strong joint panel zones will see the damage occurring in the beams at the joints. The damage level of a joint was assigned the larger damage index of the two damage indices of the beams framing into the joint, which was

$$DI_{joint} = \max \{DI_{i,} DI_j\} \tag{3}$$

where DI_i is the damage index for the beam connection left to the joint, and DI_j is the damage index for the beam connection right to the joint.

Story Damage Index. The damage indices of all joints in a story were then weight–averaged by assigning importance factors (weights) to obtain the damage index for the story. The importance factors were determined from a serviceability aspect, where the importance factor for a joint was defined as the relative importance of the joint on the lateral stiffness of the overall story where the joint is located. It was achieved by applying a lateral force system similar to the force distribution obtained through the code prescribed direct seismic design procedure. The formula for calculating a story damage index was

$$DI_{story} = \frac{\Sigma\ d_i\ DI_{joint\ i}^{\ k+1}}{\Sigma\ d_i\ DI_{joint\ i}^{\ k}} \tag{4}$$

where d_i is the importance factor for joint i, and the influences of the more severely damaged joints to the story damage are taken into account by the exponent k. The normalized damage index required the weights sum to unity.

Frame Damage Index. Based on the assumption that all stories as the structural components have the equal importance to make the frame a functional structure, a very simply weighting system could be employed for a frame with n stories: the importance factor for the first story is *1*; the importance factor for the top story is $1/n$; and the values of importance factors for all other stories vary linearly between *1* and $1/n$ along the height of the frame. This linear relationship is shown in Fig. 6. The normalized importance factor for the i_{th} story, w_i, was

$$w_i = \frac{2(n + 1-i)}{n(n + 1)} \tag{5}$$

Therefore, the overall damage index for the frame was

$$DI_{frame} = \frac{\Sigma\ w_i\ DI_{story\ i}^{\ (k+1)}}{\Sigma\ w_i\ DI_{story\ i}^{\ k}} \tag{6}$$

where the exponent k has the same meaning as discussed for the above equations and the weights should sum to unity.

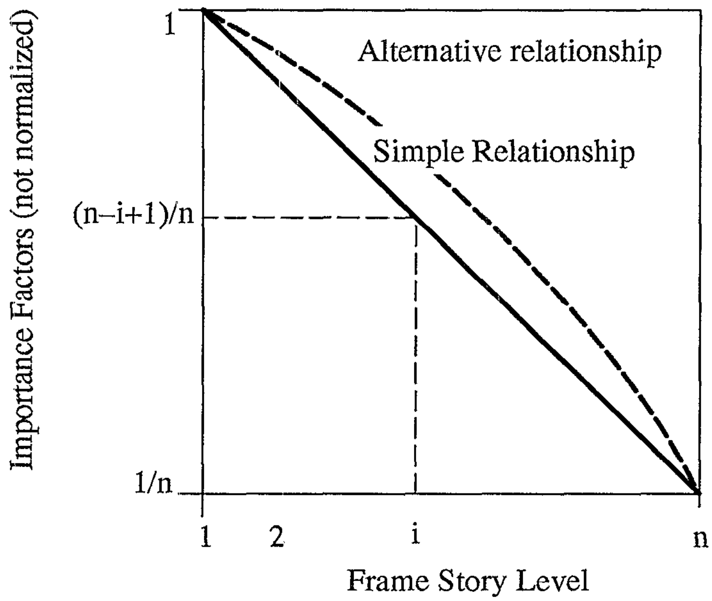

Fig. 6. Importance Factor for Different Stories

Depending upon the use of the damage index, the relationship between the story level and the importance factor might deviate from the assumed linear relationship. Possibly, the more realistic calibration between the importance factor and a story level will be something like the curve shown in Fig. 6. The determination of this relationship should be based on actual earthquake damage and full–scale testing. The systematic damage assessment approach proposed here for steel structures is similar to the approach for reinforced concrete structures presented by Bracci *et al* (1989). The difference between the two approaches is in the determination of importance factors for the joints and stories. Bracci *et al.* determined the importance of a joint or a story according to the total tributary gravity loads it carried. The results from the two set of approaches was compared to each other, and found to be similar.

CONCLUSION AND SUMMARY

Analytical investigations were performed on both cyclically loaded modified connections as well as a MRF with modified connections subjected to six California earthquakes. The main findings were:

1) The stiffness of a modified connection changed due to the modification was very small;
2) With 10% reduction in the bending capacity of a beam section at a distance *d/2* from the beam–column interface of the connection, about 20% reduction in the extreme fiber strain at the beam–column interface during a large inelastic deformation was achieved; and
3) Comparison of the analytical results between a original MRF and a MMRF showed almost no changes in the maximum story shears and story drifts induced by various California earthquakes.

The results from this research indicated that the use of the proposed modified connections in a steel moment resisting frame can be a feasible solution to mitigate seismic damage. One observation about using the proposed modified connection was: in spite of the advantages the modified connections have, the reliability of a modified WFBW connections would ultimately depend on the quality of the welding process.

For the damage assessment of steel structures, a hysteretic energy based damage model for moment resisting connections was developed. The connection damage model related the damage level of a connection to the hysteretic energy dissipated in the connection, which was realized as the direct cause of structural damage. The systematic damage assessment approach, based on this damage model, made it possible to assess the seismic damage of a steel MRF quantitatively. An important future research associated with the overall steel building damage evaluation is to find the range of damage indices which will be assigned to different damage levels, such as collapsed, irreparable, repairable, adequate for normal functioning. The overall damage assessment approach can be used in many aspects, such as 1) the design of new buildings, where through the use of a damage index, the prediction of the accumulated damage during cyclic deformation can be applied to form new design criteria and the new design criteria will account for damage potential of the frame; 2) retrofit decision making prior to earthquake events, where the level of protection can be determined through analysis, and therefore, appropriate decision could be made to retrofit a structure to resist future earthquakes; 3) post–earthquake assessment, where after an earthquake, the condition of a damaged structure can be evaluated to determine if the damage is severe enough to warrant demolition of the structure or could some repairs be made so that the structure is adequate for future use.

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