APPRAISAL OF SIMULATION TOOLS FOR FRACTURE IN STEEL COMPONENTS UNDER SEISMIC LOADING

A.P. Tola¹, I. Koutromanos² and M.R. Eatherton²

ABSTRACT

This paper assesses the capability of an existing damage accumulation law to predict fracture due to ultra-low-cycle fatigue in steel structures under earthquake loading. Finite element analyses are conducted for three experimentally tested steel components having different stress-states governing fracture, namely, a steel plate shear wall, a beam-to-column moment connection assemblage, and a brace. All three components were subjected to quasi-static cyclic loads. The analyses reproduced the global Force-Deformation response prior to fracture initiation; however, strength degradation due to fracture was not captured. The analyses yielded satisfactory results regarding the location of fracture initiation; however the instant where fracture initiates in the analyses was delayed for stress states governed by high Lode angle parameter (i.e. the beam-column connection) and it occurred too early for stress states with widely varying Lode angle parameter (i.e. the brace).

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This paper assesses the capability of an existing damage accumulation law to predict fracture due to ultra-low-cycle fatigue in steel structures under earthquake loading. Finite element analyses are conducted for three experimentally tested steel components having different stress-states governing fracture, namely, a steel plate shear wall, a beam-to-column moment connection assemblage, and a brace. All three components were subjected to quasi-static cyclic loads. The analyses reproduced the global Force-Deformation response prior to fracture initiation; however, strength degradation due to fracture was not captured. The analyses yielded satisfactory results regarding the location of fracture initiation; however the instant where fracture initiates in the analyses was delayed for stress states governed by high Lode angle parameter (i.e. the beam-column connection) and it occurred too early for stress states with widely varying Lode angle parameter (i.e. the brace).

Introduction

Steel structures in earthquake-prone regions rely on lateral force-resisting systems such as moment frames, braced frames and plate shear walls. The ductility capacity of steel structural systems critically hinges on the occurrence of rupture due to ultra-low-cycle fatigue (ULCF). Rupture is typically initiated at component locations where severe inelastic deformations due to local buckling and subsequent tensile straining occur.

Finite element simulations and ULCF laws can provide estimations of fracture initiation in structural components; such laws typically rely on the accumulation of a scalar fatigue parameter that accounts for irreversible micromechanical processes ultimately leading to rupture. The latter is assumed to occur in a ULCF law when the fatigue parameter attains a critical value.

A variety of ULCF laws exists, depending on the stress intensity measures considered and

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the conditions governing the evolution of the fatigue parameter. A stress intensity measure often considered in ULCF laws is the stress triaxiality \( \eta = \sigma_V / \bar{\sigma} \), where \( \sigma_V \) is the volumetric stress and \( \bar{\sigma} \) is the effective (von Misses) stress. Early work on void growth for monotonic loading by McClintock [1] and Rice and Tracey [2] mathematically described the influence of the triaxiality on the fracture process of metals, and a modification to such formulations by Kanvinde and Deierlein [3] allowed their extension to ULCF. These formulations rely on the void growth concept. Additionally, Lemaitre [4] proposed in 1992 a damage accumulation law that depends mainly on the triaxiality, and modifications to such formulation were done by Dufailly and Lemaitre [5], and by Huang and Mahin [6]. The latter laws rely on continuum damage mechanics.

A different set of ULCF laws have suggested the inclusion of the third deviatoric tensor invariant, \( J_3 \), to account for damage accumulation at stress states characterized by low triaxiality or close to pure shear. To this end, an additional stress intensity measure known as the Lode angle parameter \( \xi \), where \( \xi = 27/2 \left( J_3 / \bar{\sigma}^3 \right) \), has been included in the formulation of ULCF laws. Some of these laws include the work by Cao et al [7], Smith et al [8], Kiran and Khandelwal [9], and Wen and Mahmoud [10].

The purpose of this study is to evaluate – through finite element analyses of structural steel components under cyclic loading – the capability of the ULCF law by Huang and Mahin [6] to accurately describe fracture initiation and propagation in three structural components previously tested under cyclic loading. The analyses are accompanied with an investigation of the history of triaxiality and Lode angle parameter in fractured elements of each component.

**Description of Analyses**

This section briefly describes the experimentally tested components and the analysis scheme for the finite element analyses pursued in the present study.

**Components used for validation analyses**

The validation analyses were conducted for three experimentally tested structural components shown in Fig. 1, namely, a solid steel plate shear wall tested by Phillips [11], a beam-to-column moment connection tested by Eatherton et al [12], and a steel brace tested by and Fell et al [13]. The use of three different structural components is motivated by the need to evaluate the predictive capabilities of the selected ULCF law for a range of stress states ultimately leading to rupture.

![Figure 1](image-url). Components used for validation analyses. (a) Steel plate shear wall tested by Phillips [11], (b) Beam-to-Column Moment Connection tested by Eatherton et al [12], (c) Steel Brace tested by Fell et al [13].
Finite element modeling scheme

The finite element analyses were conducted in the commercial software LS-DYNA [14]. An explicit central-difference scheme was used for the global equations of each model. Shell elements, based on the formulation of Belytschko and Tsay [15], [16] and having five integration points along the thickness, were used for the modeling of the shear wall and the brace. The analysis of the beam-to-column moment connection used continuum elements to capture the progressive rupture (if any) along the thickness of the flanges or web from the beam and column. Some characteristics of these models are shown in Fig. 2.

A constitutive model with combined isotropic and kinematic hardening was used for the finite element simulations; the governing hardening law is explained in detail in [6] and [14], but it has been partially repeated here for convenience. The yield stress, \( \sigma_y \), and the rate of the back stress tensor, \( [\dot{\alpha}] \), are given by the following expressions:

\[
\sigma_y = \sigma_{y0} + \frac{H}{\beta} \left[ 1 - \exp \left( -\beta \cdot \ddot{\varepsilon}^p \right) \right] \\
[\dot{\alpha}] = \frac{2}{3} C \left[ \ddot{\varepsilon}^p \right] - \gamma [\alpha] \ddot{\varepsilon}^p
\]

where \( \sigma_{y0} \) is the yield stress at zero plastic strain, \( H \) is the isotropic hardening modulus, \( \beta \) is the isotropic hardening parameter, \( C \) is the kinematic hardening modulus and \( \gamma \) is the kinematic hardening parameter [14]. Table 1 specifies the hardening variables for each component, which were taken or slightly modified from [11], [17], and [18].

Table 1. Material hardening parameters

<table>
<thead>
<tr>
<th>Component</th>
<th>( \sigma_{y0} ) (ksi)</th>
<th>( H ) (ksi)</th>
<th>( \beta )</th>
<th>( C ) (ksi)</th>
<th>( \gamma )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Wall</td>
<td>43.3</td>
<td>0.0</td>
<td>0.00</td>
<td>47</td>
<td>0.0</td>
</tr>
<tr>
<td>Moment Connection</td>
<td>52.6</td>
<td>28.0</td>
<td>4.00</td>
<td>390</td>
<td>11.5</td>
</tr>
<tr>
<td>Brace, Flats</td>
<td>68.0</td>
<td>60.0</td>
<td>6.00</td>
<td>300</td>
<td>25.0</td>
</tr>
<tr>
<td>Brace, Corners</td>
<td>73.0</td>
<td>76.1</td>
<td>5.25</td>
<td>850</td>
<td>160.0</td>
</tr>
</tbody>
</table>
Description of ULCF law selected for the analyses

The material behavior of the steel is described in the analyses using the damage-plasticity constitutive model by Huang and Mahin [6], wherein the damage parameter $D$ evolves in accordance with the following ULCF law

$$
\dot{D} = \left( \frac{Y}{S} \right) \frac{\dot{\varepsilon}_p}{\varepsilon} \quad \text{for} \quad \eta > -\frac{1}{3}
$$

(3)

where $S$ and $t$ are constant parameters ($S$ has units of stress and $t$ is dimensionless), $\dot{\varepsilon}_p$ is the rate of the effective plastic strain, and $Y$ is the internal energy density release rate [4], which depends on the local stress state of the material. For an isotropic, linearly elastic material, $Y$ is given by the following expression [4].

$$
Y = \frac{\sigma^2}{2E} \left[ \frac{2}{3} (1 + \nu) + 3(1 - 2\nu) \eta^2 \right]
$$

(4)

where $E$ is the modulus of elasticity, and $\nu$ is the Poisson’s ratio. Material failure, i.e. rupture, is assumed to occur when the damage parameter attains a critical value, $D_c$. Rupture is accounted for in the analyses through element removal; specifically, whenever the rupture criterion is satisfied for either quadrature point of an element, the element is deleted from the model.

The employed ULCF law requires the calibration of three variables, namely, $S$, $t$, and $D_c$. Huang and Mahin [6] suggested using a value of $S$ between $F_y/200$ and $F_u/200$. The present study uses a value of $F_y/200$ for $S$. Additionally, parameters $t$ and $D_c$ are set equal to 1.0 and 0.5, respectively, based on recommendations provided in the analysis software documentation [14]. The specific values of $S$, $t$, and $D_c$ will be referred to as “default values” for the remainder of this paper.

Analysis Results

This section compares the results of the finite element simulations of the three structural components with the corresponding experimental observations. The comparison is focused on the global force versus deformation response, the instant and location of fracture initiation, and the crack propagation path. Information regarding the stress states of fractured elements from each component is also presented.

Global Force-Deformation Response

The global force-deformation response from the experiments and analyses is shown in Fig. 3. In general, the analyses of all components captured well the pre-fracture global Force-Deformation response; however, they did not capture the strength degradation due to fracture.

The shear wall experiment showed strength degradation in the last three cycles (Fig. 3a); however, the analysis did not show any strength loss. In the case of the beam-to-column moment connection (Fig. 3b) the strength loss in the experiment and analysis is due to local buckling of the flanges at the reduced beam section (RBS), and not due to fracture. Strength degradation in this
component accelerated at the last cycle of the experiment, and that behavior was not captured by the analysis. Regarding the brace (Fig. 3c), premature strength degradation was observed.

![Fracture initiation](image)

**Figure 3.** Global Force vs. Deformation response. (a) Shear Wall, b) Beam-to-Column Moment Connection, c) Brace.

**Fracture initiation**

The instant of fracture initiation from the experiments and from the analyses is shown in Fig. 4, using the applied displacement histories as reference.

Close agreement between experimental and the analytically obtained instant of fracture initiation was observed for the shear wall (Fig. 4a), with a difference of one cycle; however, a difference of four cycles of large amplitude was observed for the case of the beam-to-column moment connection (Fig. 4b). In the case of the brace (Fig. 4c), fracture initiation from the analysis occurred two cycles before the reported from the experiment.

![Fracture initiation](image)

**Figure 4.** Instant for fracture initiation. (a) Shear Wall, b) Beam-to-Column Moment Connection, c) Brace.

**Location of fracture initiation**

The analytically obtained fracture initiation location of each component is shown in Fig. 5. In the case of the shear wall, fracture initiation was obtained in the vicinity of the bottom right corner of the wall, at the gap between the vertical and horizontal edge plates. In the experimental test, fracture initiated at the bottom left corner. This difference is considered minor because the analysis gave a similar damage accumulation at both bottom corners of the wall, and the actual location
during the experiment could have been triggered by material imperfections on the bottom left corner.

Figure 5. Fracture initiation location. (a) Shear Wall, b) Beam-to-Column Moment Connection, c) Brace.

In the analysis of the beam-to-column connection, fracture initiation was obtained near the tip of the beam’s bottom flange, and along the inside face of the local buckle. Fracture in the experiment initiated in the top flange. The analysis showed similar levels of damage accumulation on the inside face of the local buckle for both flanges. Therefore, the location of fracture initiation obtained from the analysis is considered satisfactory.

For the case of the brace, the location of fracture initiation was obtained at the middle of the brace, at one of the corners, and within a region experiencing local buckling; such location is similar to the one observed from the experiment.

**Crack propagation**

The progression in the removal of fractured elements in the analyses accounts for the crack propagation during the experiments, and the results from the analyses are presented for different values of drift ratio $\theta$ in Fig. 6. The figure also shows pictures of the experimentally obtained fracture at the end of each experimental test.

The element removal sequence in the four corners of the shear wall is shown in Fig. 6a, and such sequence resembles partially the experimentally observed crack propagation. The path of removed elements tended to become aligned with the horizontal intersection of the top and bottom edge plates and the wall. In the experiment, the cracks at the corners tended to attain an inclined in-plane orientation, gradually propagating towards the center of the wall. Additionally, the length of the analytically obtained region with removed elements at the bottom left corner was approximately equal to 9 in., while the length of the corresponding crack in the experimental test was equal to 18 in. These discrepancies between analysis and experiment may be due to the fact that the exact effect of a propagating crack may not be accurately captured by removing entire elements whose edges are aligned with the boundary edges of the wall.

For the case of the beam-to-column moment connection (Fig. 6b), fracture on both flanges at the inside face of their local buckles started almost simultaneously; then a few additional elements were gradually removed from the inside face of the local buckle of each flange. Further removal of fractured elements was not captured by the analysis because the applied displacement history was completed slightly after fracture initiation was obtained. Additionally, at the end of the analysis, element removal was also obtained near the intersections of the flanges with the web,
which was not consistent with observations from the test.

The analytically obtained zone of removed elements for the brace had a similar geometry to that of the experimentally observed crack; however, the fracture propagation in the analysis was more gradual than in the experiment, in which fracture happened almost immediately after fracture initiation.

![Fracture initiation in analysis](image1)

Fracture initiation in analysis

End of 2nd cycle, \( \theta = 3.0\% \)

End of 2nd cycle, \( \theta = 5.0\% \)

End of analysis

Fracture in experiment

![Fracture initiation in analysis](image2)

Fracture initiation in analysis

Peak before the end of 2nd cycle, \( \theta = 4.7\% \)

End of analysis

Fracture in experiment

![Fracture initiation in analysis](image3)

Fracture initiation in analysis

End of 2nd cycle, \( \theta = 1.85\% \)

End of 1st cycle, \( \theta = 2.68\% \)

End of analysis

Fracture in experiment

Figure 6. Element removal sequence. (a) Shear Wall, b) Beam-to-Column Moment Connection, c) Brace.

**Crack propagation using alternative values of \( S, t, \) and \( D_c \) matching the instant of fracture initiation from experiment**

To further study crack propagation, alternative values of \( S \) and \( t \) were calibrated to match fracture initiation from the experiments. The parameters \( S = F_y/200 \), and \( D_c = 0.5 \) were maintained for the shear wall and the brace, and the respective calibrated values of \( t \) were \( t = 1.10 \) and \( t = 3.00 \). For the beam-to-column moment connection, the parameters \( S = 0.045 \) and \( t = 1 \) matched fracture initiation.

In the case of the beam-to-column connection, fracture initiation was accompanied by an almost immediate fracture across the top flange and even fracturing the web, indicating faster damage accumulation compared to the experiment. On the other hand, the reverse happened on the brace, where rupture occurred approximately three cycles after fracture initiated, indicating a slower damage accumulation post-fracture compared to the experiment.

From the results of this and the previous sections, it seems that one set of fracture parameters is incapable of capturing fracture initiation and crack propagation in all the three structural components used in this study.
Triaxiality and Lode angle parameter for fractured elements

To further understand why the fracture model may have difficulty capturing the fracture initiation in all three models, the triaxiality and Lode angle parameter obtained from the analyses for several fractured elements at all the steps with damage accumulation, is shown in Fig. 7. The information from Fig. 7a was extracted from fractured elements from the four corners of the wall, while the information from Fig. 7b corresponds to fractured elements from the top and bottom flanges of the beam. The equation describing the relation between triaxiality and Lode angle parameter for the case of plane stress [19] is also shown for reference in Fig. 7.

Figure 7. Triaxiality and Lode angle parameter in several fractured elements of each component. a) Steel Plate Shear Wall, b) Beam-to-Column Moment Connection, c) Brace.

Distinct regions in the triaxiality-Lode angle parameter space are covered by the fractured elements of each structural component, as shown in Fig. 7. In the case of the shear wall (Fig. 7a), the triaxiality and Lode angle parameter occupied a wide range of space on and outside the plane stress condition. The stress states for the beam-column connection (Fig. 7b) were characterized by values of $\eta$ between 0.3 and 0.5, and values of $\xi$ between 0.8 and 1.0. For the brace (Fig. 7c), stress states corresponded primarily to the plane stress condition, although other stress states are also shown.

The selected ULCF law does not account specifically for the Lode angle parameter in its formulation and that may be a limitation given that Fig. 7 shows stress states for damage accumulation with low triaxiality and varying Lode angle parameter. Other studies have shown that damage accumulation in the presence of low triaxiality is a function of the Lode angle parameter [7], [8], [9], [10].

Conclusions

Finite element simulations were conducted for three experimentally tested structural components, namely, a plate shear wall, a beam-to-column moment connection and a brace, to evaluate the capability of an existing ultra-low-cycle fatigue (ULCF) law to capture the occurrence and evolution of rupture, for a unique set of parameter values. Rupture was accounted for through an existing damage-plasticity formulation and element removal techniques. The models satisfactorily reproduced the global force-deformation response prior to fracture, and the location of rupture
occurrence, but they could not generally capture the instant of fracture initiation or the characteristics of crack propagation. The discrepancies may be due to the fact that element removal may not be capable of accurately reproducing the propagation of a crack, and that the employed ULCF law does not explicitly account for the effect of the third deviatoric invariant of the stress tensor. The latter is typically quantified through the so-called Lode angle parameter $\xi$. It was found that the three components corresponded to significantly different values of $\xi$ and to relatively low values of triaxiality.

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**References**