



Post-buckling ductile fracture analysis of panel zones in welded steel beam-to-column connections



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ABSTRACT

Experimental study on thin-walled welded steel beam-to-column connections under cyclic large plastic strain loading was conducted. Severe shear buckling in panel zones was observed during the testing. The connections have stable and excellent seismic performance, and finally failed due to the post-buckling ductile fracture. This paper aims to numerically simulate the post-buckling ductile cracking process using a proposed micro-mechanism based ductile fracture model, and further investigate critical factors, such as equivalent panel zone width-to-thickness ratio, axial load ratio and initial geometrical imperfection, that affect cracking behaviors of the connections. The cyclic ductile fracture model is verified with the experimental data, and employed to conduct the parametric analyses. Deterioration of stress-carrying capacity is also considered, so the fracture model can successfully simulate the load decrease in the load-displacement curves of the experimental results.

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1. Introduction

A number of studies on the design of panel zones in beam-to-column connections of welded steel moment-resisting frames (WSMRFs) have been conducted [1–3]. Panel zones are commonly designed strong enough to avoid premature buckling due to the shear load transferred by the adjacent beams. This design concept has been accepted by most of the countries, such as the US and China [4–6]. There are vertical connecting plates in the panel zones for spatial WSMRFs, which are used to link beam webs in the perpendicular direction. However, the beneficial effect of vertical connecting plates on the shear stability of panel zones has not been considered in current design provisions. An experimental study has been conducted on the effect of vertical connecting plates in mitigating the shear buckling of panel zones in cruciform beam-to-column connections under cyclic loading [7]. There are two types of connections in the experiments, connections with and without the vertical connecting plates. This paper focuses on the latter, where the connections failed mainly due to post-buckling ductile fracture at the base metal of panel zones.

Ductile fracture is one of the most important failure modes in metal structures, where the fracture surface is featured with the dimple pattern. This failure mode is favorable to structural engineers, because a large amount of energy can be absorbed owing to plastic straining

before ruptures of the members or structures. Metallic dampers such as shear panels and buckling-restrained braces also primarily fail due to ductile fracture. These structural components can thus absorb a large amount of energy transferred from the ground during a strong earthquake. To date, seismic performance evaluation of these structural components still highly relies on experimental studies due to the lack of convenient and accurate ductile fracture models, especially for cases when cyclic large plastic straining is involved.

A number of ductile fracture models [8–12] are proposed based on void growth theories [13,14]. Damage plasticity models termed the Gurson model [15] and the GTN model [16,17] are also widely employed to simulate ductile cracking of steel members and structures [12,18,19]. However, these models are initially developed for monotonic loading, and their applicability to cyclic loading is still open to discussion. Up to now, limited research [20–24] was carried out on ductile fracture under cyclic large plastic strain loading (CLPSL), particularly when cyclic plasticity, deterioration of stress-carrying capacity and extremely high geometrical nonlinearity are all concerned.

A two-parameter fracture model for CLPSL has been proposed [24], which consists of a crack initiation rule and a crack propagation rule. The crack initiation rule uses a semi-empirical formula based on both the void growth concept and previous experimental findings [25], where a ductile metal is postulated not to fracture when the stress triaxiality is less than a cut-off value of $1/3$ [20]. The crack propagation rule employs an energy approach, where the required energy to open a unit area crack is assumed to be a material constant, and can be determined

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using standard V-notched Charpy impact tests [24]. The fracture model works well for three types of structural steel under cyclic combined shear and normal stress loading. This paper aims to further validate the model using two full scale welded steel cruciform beam-to-column connections under CLPSL.

In this paper, the fracture model and the experimental results are first briefly introduced. Subsequently, numerical analyses of the specimens are conducted to calibrate the numerical models, fracture model and the plasticity model. Finally, the failure mechanism of the specimens is explained based on the numerical results, and further parametric studies are conducted to investigate the factors that have not been comprehensively studied in the experiments. The effects of equivalent panel zone width-to-thickness ratio, axial load ratio and initial geometrical imperfection on seismic performance, crack initiation and propagation of the connections are investigated.

2. Two-parameter ductile fracture model for cyclic large strain loading

A ductile fracture initiation rule for CLPSL was proposed based on the void growth concept and former experimental findings, where the damage index is defined in incremental form due to the stress triaxiality variation during a common loading history [24]. The damage index for crack initiation, D_{ini} , is defined

$$dD_{ini} = \begin{cases} \frac{d\epsilon_{eq}^p}{\chi_{cr} \cdot e^{-\frac{3}{2}T}} & T \geq -1/3 \\ 0 & T < -1/3 \end{cases} \quad (1)$$

where χ_{cr} is a material constant, $d\epsilon_{eq}^p$ is the incremental equivalent plastic strain, and T is the stress triaxiality. χ_{cr} can be obtained through numerical simulation of the corresponding tension coupon test according to the procedure given in the literature [11], where the post-necking modification of the true stress-true strain data is required. Ductile fracture initiation of a material is postulated to occur when D_{ini} reaches one unit. Herein, the term “ductile fracture initiation” denotes micro-crack initiation at a scale of 0.01 to 0.1 mm, which cannot be visually observed during an experiment. Meanwhile, macro-crack initiation is defined at the instant when the crack length is larger than 1 mm, and is termed as “crack initiation” in this study.

A crack propagation rule based on an energy balance approach has also been proposed in a previous study [24]. The crack propagation index, D_{prop} , is defined in the following formula,

$$D_{prop} = \frac{G}{G_c} \quad (2)$$

where G is the current absorbed energy of a unit area since the instant of fracture initiation, and G_c is the threshold value for absorbed energy to open a unit area crack. G_c is postulated to be a material constant, and a simple method to obtain it is proposed using a conventional coupon under monotonic tension at a quasi-static speed. The deterioration of stress-carrying capacity is also considered based on an effective stress concept, where the current effective stress, σ_e , is defined

$$\sigma_e = (1 - D_{prop}) \cdot \sigma \quad (3)$$

The loading and unloading tangent modulus of a damaged material, E_d , is reduced to

$$E_d = (1 - D_{prop}) \cdot E \quad (4)$$

where E is the Young's modulus of undamaged materials.

3. Experimental study on welded beam-to-column connections

3.1. Configuration of specimens

The two specimens with the configuration shown in Fig. 1 are made of Chinese Grade Q345B steel, a low alloy structural steel with a nominal yield stress of 345 MPa. The columns and beams are all H-shaped sections welded of steel plates. The nominal and measured sizes are listed in Table 1. The beams were welded to the column using complete joint penetration (CJP) welds. Three coupons from each plate thickness were tested to obtain the material properties. The average measured material properties of each steel plate are given in Table 2.

In both the American and Chinese seismic design provisions of steel structures, the following requirements are proposed to avoid premature shear buckling of the panel zone:

$$\begin{cases} t_{pz} \geq (h_{pz} + b_{pz})/90 \\ h_{pz} = h_{wb}, \quad b_{pz} = h_{wc} \end{cases} \quad (5)$$

where h_{pz} and b_{pz} are respectively the depth and width of the panel zone; h_{wb} and h_{wc} are depths of the beam and column webs. An equivalent panel zone width-to-thickness ratio, λ_{pz} , is defined according to Eq. (5) in this study.

$$\lambda_{pz} = (h_{pz} + b_{pz})/t_{pz} \quad (6)$$

In this study, λ_{pz} of the two specimens H1n3 and H2n3 are 92 and 118. Specimen H2n3 is susceptible to premature shear buckling.

Weak panel zones are designed for the two specimens to drive plastic deformation in the panel zones in this study. The ratios between panel zone plastic shear strength and the beam plastic flexural strength, η , of H1n3 and H2n3 are respectively 0.45 and 0.43. Therefore, the plastic deformation will mainly develop in the panel zones without apparent plastic straining at the beam ends. Moreover, in order to investigate the effect of the column axial load ratio, n , on the seismic performance of panel zones, the nominal n of the specimens are designed as 0.3, and the actual measured values are 0.29 due to the negative manufacturing tolerance of the plate thickness.

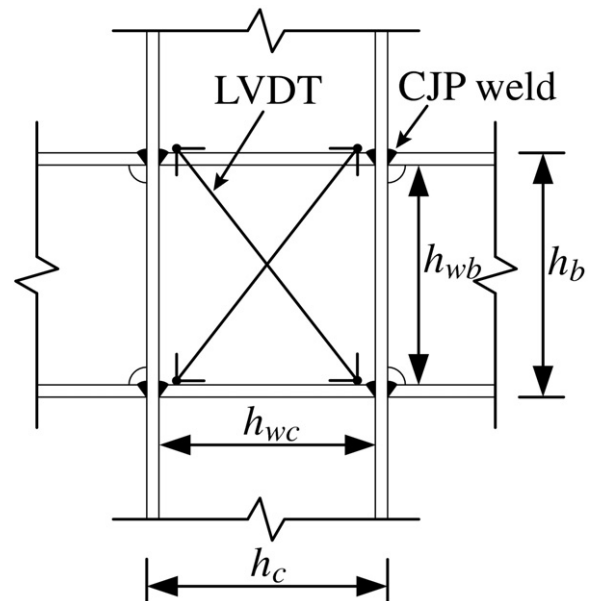


Fig. 1. Configuration of specimens at connections.

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