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Probabilistic damage assessment of concentrically braced frames with built up braces

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ABSTRACT

Concentrically braced frames (CBFs) are commonly used as seismic force resisting systems in steel building frame structures. Brace fracture is considered as one of the most common failure modes in CBFs. In this paper diagonal concentrically braced frames with braces composed of double channels stitched toe to toe are studied under cyclic loading. In this regard, a probabilistic approach is adopted to assess brace fracture. To achieve this goal, a set of past experiments was simulated numerically using Finite Element (FE) models and probabilistic damage measures were derived concerning brace fracture. Then, 27 CBFs with different slenderness ratios, various gusset plate details and connector distances were designed and analyzed numerically to assess the frame performance. Based on this assessment fragility curves for this type of bracing were generated. In order to incorporate the effects of uncertainties associated with brace, gusset plate and weld yield stress plus gusset plate thickness, two sets of data, each comprising 300 random numbers for these four variables, were generated and assigned to the designed models and the performance of frames was re-evaluated. The results show that braces with lower slenderness ratio and smaller connector distance have the least drift capacity. On the other hand, braces with high slenderness ratios and high compactness tend to withstand large drifts before fracture. The outputs also suggest that the considered model uncertainties do not have significant effect on the probability of brace fracture.

1. Introduction

Concentrically braced frames (CBF) are commonly used as seismic force resisting structural systems in buildings. Due to their relatively large stiffness and strength, they are considered as economic systems. Based on their details and the requirements they meet, CBFs may perform differently in earthquakes. AISC [1] introduces two categories for designing CBFs known as special concentrically braced frames (SCBF) and ordinary concentrically braced frames (OCBF). There are some common criteria, which shall be met in both of these two categories, as well as some additional requirements for SCBFs in order to ensure more ductile behavior. The main requirements which should be met in SCBFs consist of I) using highly ductile sections for braces, beams and columns, II) accommodating brace buckling through providing flexural strength in gusset plates or enabling the rotation of brace ends, and III) avoiding tension only braces. In both OCBFs and SCBFs, it is required that the brace connections be designed based on the brace strength, albeit some different details are prescribed for these two categories. Experimental research in the past, suggests failure in different components depending on the details of CBFs. The results of tests conducted on SCBFs indicate that in most cases, large out of plane deformation of braces due to large drift, eventually leads to local buckling in the middle of the brace. In the wake of this phenomenon, brace tearing and consequently complete rupture of the brace occur [2]. Although fracture is never desirable, this failure mode is deemed to be ductile since it allows high drift capacity before failure. On the other hand, some other experimental tests on CBFs, which do not necessarily comply with SCBFs, show that premature failure in a gusset plate connecting beam and column, due to weld tearing, is quite probable. Fig. 1 shows the CBF failure modes [3]. Gusset plate rotation due to out of plane deformation of brace imposes high demand on the corner of the gusset plate triggering interface weld crack. If the crack propagates as the drift increases, it can lead to weld rupture and finally failure of gusset plate. This mode of failure causes an abrupt drop in frame strength; therefore it is not considered as a desirable one.

A number of studies have been performed in the past, to address brace behavior including global buckling, local buckling and fracture. Parameters such as slenderness ratio, width to thickness ratio and section geometry were considered to investigate the cyclic brace behavior. A study undertaken by Trembly [4] compiled the results of 76 tests around the world in order to examine the cyclic behavior of braces including buckling strength, post-buckling behavior in different ductility levels, maximum tensile strength, out-of-plane deformation during buckling, and fracture life. It was suggested that brace slenderness is the dominant parameter affecting the seismic behavior of braces. Another research done by Fell et al. [5] explored inelastic buckling and fracture behavior of braces. The effects of various parameters such as width-thickness and slenderness ratios, cross-section shape and loading histories were investigated. Their results suggest that brace ductility is primarily a function of section compactness and to a lesser extent

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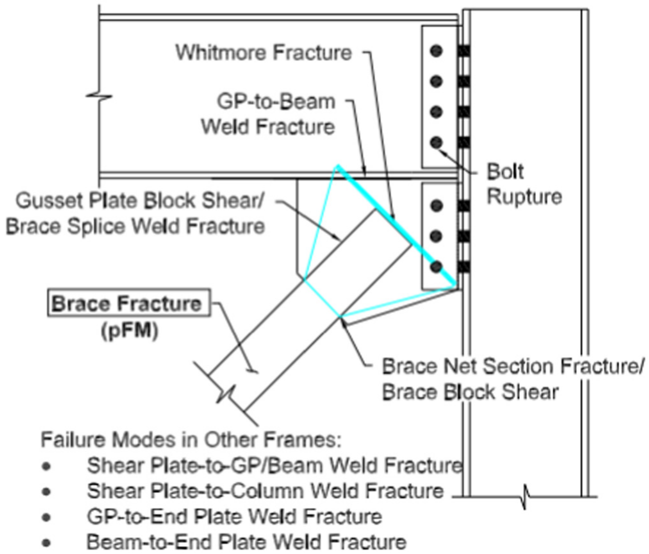

Fig. 1. Various failure modes of CBFs [3].

member slenderness and loading history. This study also indicates that pipe and wide-flange sections exhibit more gradual local buckling modes compared to HSS ones.

Gusset plate detailing is another issue, which has been interesting to researchers. Based on one of the most prominent studies in this area, Astaneh Asl [6,7,8] proposed providing a 2 t linear clearance length in the gusset plate in order to enable brace end rotation and achieve ductile behavior. Nowadays, Seismic Provisions for Structural Steel Buildings stipulates this recommendation for SCBFs as an acceptable alternative for brace buckling accommodation. Extensive research has also been done at University of Washington to explore CBF behavior. Based on the results of these studies, Lehman et al. [9] suggested a design procedure based on a hierarchical system of failure by which a desirable failure mode is obtainable. In addition, they proposed a 6 t to 8 t elliptical clearance length in lieu of a 2 t linear one.

Interface welding, which connects gusset plate to beam and column, has been another area of research. AISC [1] requires that the connection be designed based on the brace capacity while Roeder et al. [10] suggest gusset plate capacity as the basis for weld design.

The above review of CBF literature shows that although quite a few studies have been done to address CBFs, only a limited number of them considered those which use built-up sections as their braces [6,8,11]. However, AISC [1] stipulates some regulations with regard to built-up sections including distance of connectors as well as their locations.

In general it can be inferred that CBFs demonstrate various behavior (i.e. different failure mode and consequently different ductility capacity) depending on the philosophy adopted for their design. On the other hand, variation of material properties, element dimensions and constructional quality can affect the expected behavior. This study tries to quantify variation of behavior caused by variables mentioned above. In this regard, some CBFs with different characteristics reflecting design variables as well as design philosophies were selected. The frames were modeled using Finite Element (FE) software. To compare the behavior of the frames and quantify the differences, a probabilistic damage measure was defined based on the experimental tests, which have been undertaken around the world. The defined damage measure was then used to assess the CBF performance. A probabilistic approach was taken to incorporate the effects of material, dimensional and constructional uncertainties in performance evaluation. The results are finally presented as fragility curves.

2. Deriving damage measure

In order to evaluate the performance of a structure, it is crucial that certain criteria be defined. With regard to CBFs, brace fracture and interface weld tear are the two ultimate damage states that a CBF can experience. However, in the case of FE modeling, incorporating damage is challenging. It is primarily due to the fact that plasticity models are not capable of capturing phenomena such as brace fracture or weld tearing. Nevertheless, previous studies took an implicit approach to address this issue. For instance, Hsiao et al. [12], Takeuchi and Matsui [13], Lai & Mahin [14], Uriz [15] proposed some models based on cumulative strain to predict the fracture of the braces. These models, considered as macro-models, have assumed strain as the only parameter governing the fracture. They calibrated their models by experimental observations and data. More sophisticated fracture models such as cyclic void growth model (CVGM) [16] consider triaxility as well as strain to predict fracture. Although this phenomenological model is believed to be more accurate than simple ones, it warrants quite fine meshes and parameter calibration. Therefore, complicated fracture models are practically difficult for application particularly owing to their calculation cost.

Due to the limitations mentioned above, in this study a cumulative strain based approach was adopted. Hence, equivalent plastic strain or PEEQ, which is inherently cumulative and reflects strain history of an element, was selected as the fracture index.

To determine the critical value of fracture index, 14 experimental tests from different studies were considered. Table 1 provides the

Table 1
Selected experimental studies used in the data set.

Study reference	Specimens designation	Brace section	Steel type	Ag (mm^2)	w/t D/t	L/r	Loading history	Setup category	Primary failure mode
[5]	Kanvinde-1	HSS4 × 4 × 1/4	A500, Gr. B	2174	14.2	77	Far-Field (S)[a]	Bare brace	Brace fracture
[5]	Kanvinde-2	HSS4 × 4 × 1/4	A500, Gr. B	2174	14.2	77	Near-Fault (C)[b]	Bare brace	Brace fracture
[5]	Kanvinde-4	HSS4 × 4 × 3/8	A500, Gr. B	3084	8.46	83	Far-Field (S)	Bare brace	Brace fracture
[5]	Kanvinde-6	Pipe3STD	A53, Gr. B	1632	16.2	103	Far-Field (S)	Bare brace	Brace fracture
[5]	Kanvinde-9	Pipe3STD	A53, Gr. B	1632	16.2	103	Near-Fault (T)[c]	Bare brace	Brace fracture
[5]	Kanvinde-11	Pipe5STD	A53, Gr. B	3147	21.6	64	Near-Fault (T)	Bare brace	Brace fracture
[5]	Kanvinde-13	Pipe5STD	A53, Gr. B	3147	21.6	64	Far-Field (S)	Bare brace	Brace fracture
[5]	Kanvinde-19[d]	HSS4 × 4 × 1/4	A500, Gr. B	2174	14.2	77	Far-Field (S)	Bare brace	Brace fracture
[15]	Uriz	HSS6 × 6 × 3/8	A500, Gr. B	5213	14.2	49	Far-Field (S)	2-Story frame	Brace fracture
[17]	UW HSS1	HSS5 × 5 × 3/8	A500, Gr. B	3987	11.3	72	Far-Field (A)[e]	1-Story frame	GP weld rupture
[17]	UW HSS2	HSS5 × 5 × 3/8	A500, Gr. B	3987	11.3	72	Far-Field (A)[e]	1-Story frame	Brace fracture
[17]	UW HSS4	HSS5 × 5 × 3/8	A500, Gr. B	3987	11.3	72	Far-Field (A)[e]	1-Story frame	Brace fracture
[17]	UW HSS5	HSS5 × 5 × 3/8	A500, Gr. B	3987	11.3	72	Far-Field (A)[e]	1-Story frame	Brace fracture
[18]	UW HSS6	HSS5 × 5 × 3/8	A500, Gr. B	3987	11.3	72	Far-Field (A)[e]	1-Story frame	Brace fracture

[a] Applied by symmetrical far-field load protocol.
[b] Applied by compression near fault load protocol.
[c] Applied by tension near fault load protocol.
[d] Reinforced in the middle of the brace by plates.
[e] Applied by asymmetrical far-field load protocol.