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Numerical and experimental evaluation of seismic capacity of high-rise steel buildings subjected to long duration earthquakes

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ABSTRACT

Occurrences of large earthquakes having a magnitude larger than eight along subduction zones have been reported worldwide. Due to large number of load reversals the effect of cumulative damage on structural components due to deterioration becomes critical for steel buildings of old construction but may also become critical for buildings designed based on current seismic provisions. A state-of-the-art analytical model that simulates component deterioration and fracture due to low cycle fatigue has been developed and implemented in the OpenSees computational framework. The model serves for seismic evaluation of steel moment frame structures subjected to long duration records. The effectiveness of the numerical model in quantification of the seismic capacity of high rise steel structures is demonstrated through validation with a full scale shaking table test of a high-rise steel building subjected to a long duration record at the world's largest shaking table facility (E-Defense). Limitations of the proposed numerical model are also discussed.

1. Introduction

Recent earthquakes around the world (Northridge, 1994; Kobe, 1995) raised many concerns regarding the seismic performance of steel moment resisting frames (MRF) due to the occurrence of brittle fractures of welded beam-to-column connections [1,2]. Various analytical studies [3–6] summarized the effect of brittle fractures on seismic capacity of existing steel moment frames. This forced the earthquake engineering community to design improved steel connections so that brittle fracture is avoided [7,8].

However, improved steel moment connections may not be invincible when subjected to a large number of inelastic cycles due to low cycle fatigue [9–14]. This is more evident in high-rise steel buildings that are subjected to long period long duration ground motions that occur near subduction zones. The reason is that the predominant period of these ground motions range from several to 10 s, and their primary durations extend over several minutes [15–17]. These ground motions tend to resonate high-rise buildings whose fundamental natural periods are above 2 s. This observation was confirmed by recent full-scale shake table tests that represented typical high-rise construction in Japan [18]. These tests took place at the world's largest shake table facility at E-Defense. Prior numerical studies [19] on high-rise steel buildings subjected to long period long duration ground motions also confirmed the same observation.

During the past years a large number of numerical and experimental studies emphasized on the importance of fracture on the seismic capacity of steel MRF due to seismic loading. Nakashima et al. [20] investigated the effect of moment redistribution caused by beam fracture in steel MRF based on static loading. The main conclusion of this study was that sequential fractures are less likely to occur during static moment redistribution when rotations corresponding to fracture are large. Utilizing a large number of steel MRFs as part of the SAC phase II project in United States, Luco and Cornell [21] investigated the effect of connection brittle fractures on seismic drift demands of steel MRFs using an empirical analysis model to assess brittle fracture. They concluded that the effects of connection fractures are more pronounced at higher drift demand levels.

Rodgers and Mahin [22] demonstrated both experimentally and numerically that severe strength loss due to the combination of numerous fractures in steel MRFs and large excitation can have adverse consequences, including collapse. Nakashima et al. [23] investigated experimentally the effect of residual strength of a steel MRF on connection fractures. The composite effect due to slab

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on connection fractures was also assessed in this study. They concluded that after fracture of the bottom flange of a steel beam due to slab effect occurs, the residual capacity of composite steel MRFs is about 35% of the respective maximum strength at relatively large drifts. Recent full scale shaking tables on high-rise steel buildings conducted on the world's largest shaking table at E-Defense [10,18,24] demonstrated the vulnerability of steel beam-to-column connections to long period long duration ground motions. In these connections, fracture occurs due to the large number of inelastic deformations resulting to severe strength deterioration.

Consequently, there is a need to investigate the effect of component deterioration and ductile fracture on the seismic capacity of high-rise steel buildings. This will contribute to effective decisions on retrofit techniques for steel beam-to-column connections. For this purpose, a numerical model that is able to simulate complex deterioration phenomena and ultimately connection fracture due to low cycle fatigue has been developed and implemented in the Open System for Earthquake Engineering Simulation (openSees) platform [25], which is available from the Pacific Earthquake Engineering Research Center (PEER). The component model has been calibrated against a large number of steel component tests that have been conducted over the past years around the world. The validity of the numerical model is demonstrated with recently conducted full scale shaking table test of a high-rise steel building [18,24]. This test took place at the E-Defense facility in Japan. The seismic capacity of the high-rise steel structure after the occurrence of connection fractures is evaluated numerically after utilizing the same numerical model. Limitations of the proposed numerical model are also summarized at the end of this paper.

2. Numerical model for prediction of fracture due to low cycle fatigue

Various models have been proposed to investigate the effect of low cycle fatigue on steel beam-to-column connections. Bertero and Popov [26] derived a relationship between plastic strain amplitude and the number of cycles to fracture based on strain-controlled tests that lead to fracture of steel beam flanges after severe local buckling. Park et al. [27] incorporated the maximum deformation and energy dissipation due to hysteresis in a damage index. More recently, Krawinkler et al. [28] developed a numerical model that can simulate the effect of cumulative damage due to earthquake excitations based on Coffin–Manson [29] relationship considering the full loading history of a steel component subjected to cyclic loading.

For prediction of inelastic buckling and fracture of steel braces due to low cycle fatigue, a distributed plasticity model [30] that incorporates rainflow counting was developed and validated with experimental data of steel braces. Krishnan [31] developed an elastofiber element that can simulate inelastic post-buckling response including fracture of steel braces and slender columns. Kanvinde et al. [32–34] proposed and demonstrated the effectiveness of a cyclic void growth model (VGI) to assess ductile fracture initiation due to low cycle fatigue for steel connections and steel braces. Lin et al. [35] utilized the VGI model in detailed finite element studies for prediction of fracture initiation of field welded steel connections. Lee and Stojadinovic [36] developed a new cyclic yield-line plastic hinge model for estimating connection rotation capacity. This model is applicable by designers to evaluate new steel connections before required proof of performance tests. Campbell et al. [37] presented a summary of damage predictions in steel MRFs under earthquakes in which low cycle fatigue is incorporated based on a compilation of salient analytical and experimental results.

This paper focuses on the capability of simulating component deterioration and fracture due low cycle fatigue with an inelastic concentrated plasticity element that was recently implemented in the OpenSees computational framework [25] (http://opensees.berkeley.edu). In particular, a fracture rule was incorporated in the modified Ibarra–Krawinkler deterioration model [38,39] that allows modeling of (1) strength and stiffness deterioration and (2) rupture leading to complete severing of a steel beam-to-column connection due to cyclic loading. This analytical model serves for seismic evaluation of steel MRFs at extreme seismic loading, including the quantification of post-fracture response. The emphasis is on high-rise steel buildings subjected to long period long duration ground motions. Because of the large number loading reversals in these ground motions, steel connections deteriorate cyclically in strength and stiffness and ultimately fracture due to low cycle fatigue.

2.1. Component deterioration modeling

The modified Ibarra–Krawinkler phenomenological analytical model is able to simulate stiffness and strength component deterioration by using a reference backbone curve shown in Fig. 1a. To control cyclic deterioration, an energy-based rule proposed by Rahnama and Krawinkler [40] is used. The hysteretic response of the modified Ibarra–Krawinkler model has been calibrated extensively during the last few years by utilizing a steel component database for deterioration modeling. This database includes tests for more than 300 steel beams [39,41]. Fig. 1a shows an example of calibration of the elastic stiffness K_e, pre- and post-capping (after local buckling occurs) rotation θ_p, θ_{pc} of a steel beam subjected to monotonic loading. The modified Ibarra–Krawinkler model deteriorates cyclically using a deterioration parameter Λ. This parameter is a reference energy dissipation capacity of a steel component that controls strength, stiffness and post-capping strength deterioration modes (see Fig. 1b). A detailed description about the original and modified Ibarra–Krawinkler deterioration model can be found in [38,39].

2.2. Modeling of fracture due to low cycle fatigue

In order to incorporate low cycle fatigue in the numerical model discussed in Section 2.1, a fracture law was implemented. For a steel component, the number of cycles to fracture N_f is expressed as a function of cumulative energy dissipation E_d of the steel component when subjected to cyclic loading. This expression can be written according to Eq. (1),

$$N_f = a_1 E_d^{-k} \tag{1}$$

This relationship is analogous to the Coffin–Manson [29] equation for low-cycle fatigue simulation,

$$N_f = b \Delta\varepsilon_p^{-a} \tag{2}$$

where $\Delta\varepsilon_p$ is the amplitude of plastic strain of a steel component. The difference between Eqs. (1) and (2) is that instead of the amplitude of plastic strain of the steel component, the dissipated hysteretic energy is expressed with respect to the number of cycles to fracture N_f. The logarithmic expression of Eq. (1) is given by,

$$\log N_f = \log(a_1) - k \log(E_d) \tag{3}$$

Eq. (3) is further simplified by,

$$\log N_f = A - k \log(E_d) \tag{4}$$

The parameters A and k in Eq. (4) depend on the loading history that a steel component experiences as part of a structure subjected to an earthquake. Fig. 2 shows the effect of two different loading histories on the parameters A and k. The two protocols that are used are a standard symmetric loading protocol and a near-fault loading