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Seismic performance evaluation of damage tolerant steel frame with composite steel-UHPC joint

Tengfei Xu [a,c], Di Zheng [a], Cheng Yang [b], Kailai Deng [a,c,*]

[a] Department of Bridge Engineering, Southwest Jiaotong University, Chengdu 610031, PR China
[b] Department of Building Engineering, Southwest Jiaotong University, Chengdu 610031, PR China
[c] Key Laboratory of High-speed Railway Engineering, Ministry of Education, Southwest Jiaotong University, Chengdu 610031, PR China

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ABSTRACT

In order to reduce the earthquake damage and improve the resilience of frame structures, a damage tolerant steel frame was proposed in this paper. In this steel frame, the composite Ultra-High Performance Concrete (UHPC) joint and friction damper were applied at the beam to column connection. The UHPC joint can provide excellent crack resistance and deformation capacity, while the friction damper can resist the moment in normal service and dissipate energy under earthquake. To verify the seismic performance of this novel frame, four sophisticated finite element models were built by using ABAQUS, including one conventional moment resisting frame and three novel steel frames with different friction forces. Pushover analysis and nonlinear dynamic analyses were carried out. Compared to conventional frame, both deformation and base shear force of the novel frames were significantly reduced in the simulations. Under maximum considered earthquake (MCE), the conventional steel frame sustained significant yielding in the beam-column connection and column base. While for the novel frames, more than 80% external works were dissipated by the friction dampers, and only slight plastic strain appeared on the composite UHPC joint and column base, demonstrating the realization of the expected damage tolerant performance. Furthermore, the relative small friction force was suggested according to the comparisons between novel frames. The early yielding mechanism caused by the weak friction dampers can effectively improve the energy dissipation performance and damage control.

1. Introduction

Recent earthquakes, i.e. Wenchuan earthquake 2008, Tohoku earthquake 2011, Mexico earthquake 2017, caused significant damage to the buildings. The post-earthquake recovery is time consuming and costly, which results in the inevitable huge loss in society function [1, 2]. One available approach for minimizing the disturbance is developing the earthquake resilient structures, i.e. controlling the degree and occurrence of earthquake damage on the main structures, and accelerating the recovery time by introducing the replaceable energy dissipation derives [3, 4].

In moment resistant steel frames, earthquake damage mainly concentrates on the beam-column connections. The connection zone usually sustains significant local buckling at beam ends or panel yielding in the column. To eliminate the unexpected seismic damage, several novel beam connections are proposed, i.e. self-centering connections [5, 6], shape memory alloy connections [7], and damage avoiding connection [8, 9]. A series of tests and simulations proved the effectiveness of these novel connections in reducing the seismic damage. The essence of these research is the plastic hinge relocation theory, which forces the plasticity appearing on the easily replaceable energy dissipation device and maintaining the major structural components in elastic. Many nonlinear dynamic analyses are carried out to evaluate the seismic performance of these novel frame structures [10, 11, 12]. Nevertheless, most earthquake assessments on the frame structures are based on fiber models without involving the reinforced concrete (RC) slab, which could not deliver the local damage and plasticity distribution in the connection zones. Nonlinear dynamic analysis based on the sophisticated numerical model of these novel damage tolerant frames has yet to be conducted.

This paper developed a novel steel frame to achieve damage tolerant performance under maximum considered earthquake (MCE). By introducing the composite ultra-high performance (UHPC) joint and friction damper, the deformation in the connection zone can concentrate on the designated region, and most input energy will be dissipated by the friction damper. To assess the seismic performance, four sophisticated numerical models were built by using ABAQUS, including one benchmark moment resistant steel frame and three novel damage tolerant steel frames. Pushover analysis and nonlinear time history analyses were carried out at four intensities of seven ground motion records. The

* Corresponding author at: Department of Bridge Engineering, Southwest Jiaotong University, Chengdu 610031, PR China.
E-mail address: kailai_deng@163.com (K. Deng).

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deformation, plasticity development, and energy dissipation were obtained and discussed. According to the analysis results, the novel steel frames presented evident superior on the damage control, the reductions in story drift and base shear force, and the energy dissipation capacity against the conventional moment resistant frames.

2. Sophisticated numerical model

2.1. Structure design

A typical moment resistant steel frame structure with the standard span of 6 m was designed as shown in Fig. 1 [13]. The 1st story was 4 m height, whereas the other stories (2nd to 4th) were 3.5 m height. As shown in Fig. 1(b), the steel frame comprised steel columns, steel beams and reinforced concrete floors. Q345B steel with nominal yielding stress of 345 MPa and C30 concrete with the cubic compressive strength of 30 MPa were used in this frame. The dead load on the floor was 5 kN/m^2. In Beijing area, the seismic fortification intensity is 8 degree according to Chinese seismic design code [14], corresponding to the peak ground motion accelerations (PGAs) of 70 Gal, 200 Gal, 400 Gal and 600 Gal for service level earthquake (SLE), design based earthquake (DBE), maximum considered earthquake (MCE) and and very rare earthquake (VRE). In Chinese seismic design code, two stages design approach is suggested, i.e. strength demand and story drift ratio limitation of 1/250 under SLE and story drift ratio limitation of 1/50 under MCE for steel frame. In this case, the story drift ratio under SLE was the most critical condition for low-rise steel frame, resulting in the relative large column and beam sections and the small fundamental period.

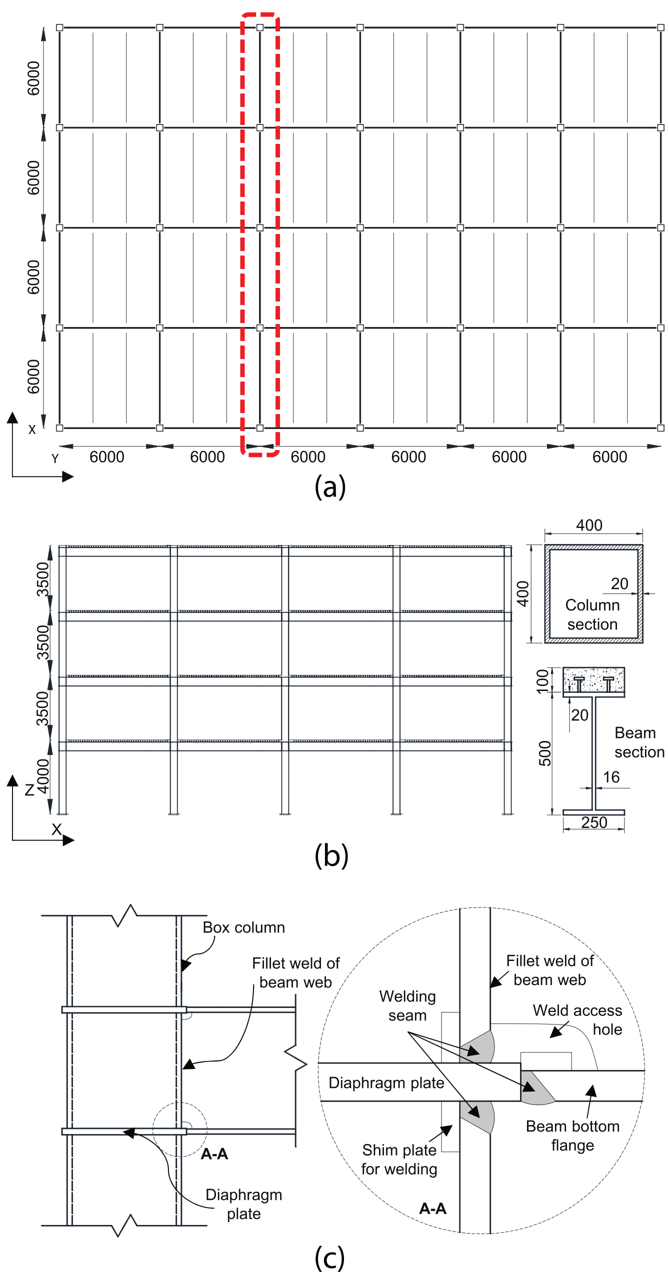

Fig. 1. Structure of conventional steel frame (a) Structural plan. (b) Structural elevation and sections of column and beam. (c) Details of connection zone.

Fig. 1(b) shows the detailed geometry and section dimensions of the conventional steel frame. The column had the box section of 400 mm × 400 mm × 20 mm, and the I-shaped beam has the height of 500 mm and flange width of 250 mm. The thicknesses of flange and web in the beam were 20 mm and 16 mm respectively. In this frame, RC slabs had the thickness of 100 mm and the reinforcement ratio of 0.3%. In the conventional steel frame, two diaphragm steel plates with the thickness of 30 mm were used in the beam-column connection, as shown in Fig. 1(c).

A novel damage free steel frame was developed through replacing the end segment of the composite steel beam in the conventional steel frame with the proposed novel connections, as shown in Fig. 2. A composite UHPC joint was introduced to connect the steel column and beam. There was a short designated plastic hinge region in this joint, which was expected to carry the most beam-column rotation. Owing to the satisfactory crack resistance of UHPC material and its small section height, the composite UHPC joint can provide extremely large deformation capacity.

The dimensions of the composite UHPC joint are shown in Fig. 2 (c). The composite UHPC joint had the same width with the beam flange. The thickness of UHPC layer was 100 mm. The designated plastic hinge region was 240 mm long. Ten studs with the dimension of φ 19–60 (diameter = 19 mm and length = 60 mm) were used to connect the UHPC layer with the steel plate, corresponding to the spacing of 120 mm for adjacent studs. This spacing can guarantee almost no shear slip occurrence at the UHPC-steel interface [15]. The top steel plate and stiffener had the thickness of 20 mm and 14 mm respectively. The UHPC layer had the thickness of 100 mm and sectional reinforcement ratio of 4%. Owing to the sufficient tensile strength, the UHPC can tolerate the 4% sectional reinforcement ratio without shrinkage induced cracking [15]. This composite UHPC joint was welded with the column, while connected to the beam through bolts.

A friction damper was employed below the composite UHPC joint to resist the moment at beam end section in normal service and dissipate energy in earthquake. The structure of the friction damper is shown in Fig. 2(d), which was consisted of two steel plates and two cast iron plates. Four movable bolts were used to apply the normal pressure between the cast iron plates and steel plates. The maximum friction force depends on the fractional coefficient and pressure at the steel-iron interface. In practice, the maximum friction force can be easily adjusted through tightening the movable bolts by using a torque wrench with a pre-calibrated torsion moment. Note that the friction force should be smaller than the yielding strength of the bottom flange in the steel beam. Thus, no local buckling or yielding will occur in the beam [9]. Another four bolts were used to fix one end of the friction damper. The permitted deformation of the friction damper depends on the distance between the two steel plates. In this case, it was 80 mm. The friction damper was connected to the column and beam through two ear plates, which can be taken as pinned connection in the plane and clamped connection out-of plane.