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Experimental study on seismic behavior of high strength steel frames: Global response

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ABSTRACT

Cyclic tests were performed on six full-scale single-bay two-story frames, including one frame using Q345 ($f_y = 345$ MPa) ordinary strength steels, and another five frames using Q460 ($f_y = 460$ MPa) high strength steels or Q890 ($f_y = 890$ MPa) ultra-high strength steels in only columns or both beams and columns. The frames were designed to provide strength and ductility for earthquake resistance with energy dissipation located at member ends by flexural yielding, and panel zones by shear yielding. Cover-plate reinforced connections were specified to relocate the plastic hinge beyond the nose of cover plates and away from the face of column to improve connection performance. As the first of two companion papers on this experimental study, this paper presents detailed procedures to design the specimens, and then provides the outline of test program including test setup, loading protocol and instrumentation. After that, global responses in experimental results, including test observations of each specimen throughout the test and their hysteresis and backbone curves, deformation and energy dissipation in each half-cycle and in total, were described. The results evidenced that satisfactory seismic behavior was identified by using high strength steel columns with compact or noncompact sections in frames, and the maximum overall drift ratio reached 4.0%. Even the frame with ultra-high strength steel columns with slender sections accommodated an overall drift ratio of 3.0%. No soft-story mechanism occurred in all the frames. Local responses in beams, columns and connections, especially in panel zones, will be examined in the second companion paper.

1. Introduction

1.1. Background and motivation

With the rapid development of new production and welding techniques, high strength steels with nominal yield strength $f_y \geqslant 460$ MPa (ultra-high strength steels with $f_y \geqslant 690$ MPa) have been used in engineering structures, due to their substantial advantages in architectural style, structural safety and economic benefit [1–4]. A comprehensive literature review in recent research advances of high strength steel structures has been conducted by the authors [5]. However, the majority of research advances in that paper focused attention on static behavior, such as cross-sectional strength, stability of beams and columns, shear behavior of panel zones and direct analysis for frames made of high strength steels under static load [6–23]. Quite few studies have been found with respect to seismic behavior, since multi-story frame structures using high strength steel members represent an innovation in current seismic design. As the first step to investigate seismic behavior of high strength steel structures, Huang et al. [24,25], Shi et al. [26,27], Wang et al. [28], Dusicka et al. [29] and Miyazaki et al. [30] carried out cyclic loading experiments of high strength steel materials and weld connections, so that constitutive models could be calibrated to describe accurately their nonlinear cyclic plasticity. Then, with respect to strength, ductility and energy dissipation capacity of structural members involving high strength steels, Ricles et al. [8], Green [31] and Suzuki et al. [32] conducted a series of cyclic loading experiments on welded I-shaped beams made of HSLA-80 steels ($f_y = 552$ MPa) and high strength steels with tensile strength equal to 590 MPa, respectively, and carried out finite-element analyses to evaluate the influence of material properties, cross-section geometry and applied loading; Dusicka et al. [33]

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experimentally examined cyclic response of HPS485 W steel (f_y = 485 MPa) shear links and satisfactory results were obtained showing a potential advantage of low overstrength; Fukumoto and Kusama [34], Kuwamura and Kato [35], Shi et al. [36,37],Wang et al. [38] and Chen et al. [39], respectively, investigated cyclic behavior of columns under compression or combined compression and bending, which were made of various high strength steels including HT80 steels (f_y = 700 MPa) made in Japan, Q460 (f_y = 460 MPa) and Q690 (f_y = 690 MPa) steels produced in China. Lin et al. [40,41] proposed a new kind of built-up columns made of H-SA700 steel plates (f_y = 700 MPa) by high strength bolts and investigated their flexural performance by bending test, and then developed corresponding bolted beam-to-column connections for such built-up columns. Sato et al. [42] developed weld-free beam-to-column connections for H-SA700 steel members using knee brace damper and friction damper to dissipate energy. Sun et al. [43] also examined cyclic behavior of beam-to-column bolted connections using Q690 steel in columns and end plates. Furthermore, Matsui and Mitani [44] conducted an early experimental study on inelastic cyclic behavior of SM58 steel (f_y = 470 MPa) frames subjected to constant vertical and alternating horizontal forces. Recently Tenchini et al. [45,46] and Dubina et al. [47,48] combined the use of high strength steels in non-dissipative members and mild carbon steel in dissipative zones, and lots of work were contributed to assess seismic performance of those dual-steel structures by experiments on structural components and analyses on frame buildings. In Japan, damage-free design method [49] has been proposed to greatly facilitate utilization of high strength steels and experimental studies by Shinsai et al. [50], Nakai et al. [51], Takeuchi et al. [52,53] were carried out on frames using H-SA700 steels in beams and columns and mild steels like SN400, SS400 in knee braces, concentric braces, or buckling-restrained braces, so that those high strength steel members remained elastic even under the extremely rare earthquakes.

The previous research clearly evidenced the effectiveness and advantages by using high strength steels in structural members desired to remain elastic under earthquakes, such as in columns in moment frames or both beams and columns in braced frames [54]; however, there is still a gap before high strength steels can be widely used in practice, since no systematic seismic design methods associated with such steels have been proposed in current codes. Even additional rules [55] to extend Eurocode 3 [56] up to steel grades S700 (f_y = 700 MPa) are questionable due to a lack of substantial research background. Therefore, comprehensive investigation on high strength steel structures, especially full-scale experimental studies, are in urgent need in order to evaluate their system performance under strong earthquakes, and to develop safe and reliable seismic design guidelines for such steel structures.

1.2. Objective

This paper presents a full-scale cyclic test program to study seismic behavior of single-bay two-story high strength steel frames, including two homogenous steel frames using Q460 (f_y = 460 MPa) steels in both beams and columns, two hybrid steel frames using Q460 (f_y = 460 MPa) steels in columns and Q345 (f_y = 345 MPa) steels in beams, and one hybrid steel frame using Q890 (f_y = 890 MPa) steels in columns and Q345 (f_y = 345 MPa) steels in beams. Another homogenous ordinary strength steel frame using Q345 (f_y = 345 MPa) steels has also been designed as the dummy specimen. Firstly, detailed design procedures and test program are illustrated in Sections 2 and 3, respectively. Then, observations throughout the test are described in Section 4. System global responses including strength, deformation and energy dissipation capacity are demonstrated in Section 5. Finally, several practical implications associated with the safety of current seismic design are discussed in Section 6 by exploring test results, and conclusions are summarized in Section 7. Detailed local responses in beams, columns and connections, in particular, panel zones, are introduced in the companion paper. This study will provide significant research basis for the new *Design Specification of High Strength Steel Structures* [5] being codified in China.

2. Design of frame specimen

2.1. Prototype building

Based on the constraints imposed by the laboratory, six specimens were designed, constructed, and tested, each of which consists of a single-bay two-story moment frame. In order to relate these specimens to an actual structure, the specimens were designed to be extracted from a six-story prototype building. Fig. 1(a) and (b) shows the plan and elevation of the prototype building, which was assumed to be located on stiff soil in Beijing, China. The typical bay span and story height were 6 m and 2.7 m in both directions. The characteristic dead (D) and live (L) loads were 6 kN/m^2 and 2 kN/m^2, respectively, for the floors and the roof. Effective seismic weights were taken as a combination of all dead loads and half of live loads (D + 0.5L), and were 4536 kN for the floors and roof, resulting in a total seismic weight of the building equal to 27,216 kN. The design followed Chinese *Code for design of steel structures* [57] with the seismic force stipulated in Chinese *Code for seismic design of buildings* [58]. The response spectrum adopted for seismic design is shown in Fig. 1(c), where the maximum response acceleration (α_{max}) depends on the earthquake level (i.e., frequently occurred earthquake FOE, design basis earthquake DBE, or maximum considered earthquake MCE), and the characteristic period (T_g) is 0.35 s depending on the location.

2.2. Member design

Beams and columns were made of welded wide-flanges in the shop, as shown in Fig. 2(a). Taking into account the lateral restraint provided by the slab in the actual building, the beams were verified as having sufficient flexural strength under three load combinations considered for ultimate limit state, i.e., 1.2D + 1.4L, 1.35D + 0.7 × 1.4L, and D + 0.5L + ΩE_{hk2}, where E_{hk2} was the seismic load corresponding to DBE defined in Fig. 1(c) and Ω was structural characteristic factor (i.e., D_s in Japanese code [59], the reciprocal of behavior factor q in Eurocode 8 [60] or response modification factor R in US code [61]) depending on the ductile behavior of structures. A recommended value of Ω = 0.4 in [57] was used. The flexural strength of beams was checked as [57],

$$\frac{M_b}{W_b} \leqslant f_b \tag{1}$$

where M_b is the maximum bending moment under any load combination, W_b is the moduli of the cross-section of beams, and f_b is the design strength of beam flange materials accounting for the partial factor γ_R, i.e. $f_b = f_{yb}/\gamma_R$ where f_{yb} is the yield strength of beam flanges.

The columns in each floor were verified in combined compression and bending under the same two gravity load combinations (1.2D + 1.4L and 1.35D + 0.7 × 1.4L), and another amplified seismic load combination, i.e., D + 0.5L + $1.1\eta_y\Omega^a_{i,\min}E_{hk2}$, where η_y is material overstrength factor (taken as 1.1) taking into account the possibility that the actual yield strength of steel is higher than nominal yield strength, and $\Omega^a_{i,\min}$ is the actual minimum characteristic factor of the i-th floor which is determined by,

$$\Omega^a_{i,\min} = \min_{\text{all beams in } i\text{-th floor}} \{(W_b f_y - M_{GE})/M_{Ehk2}\} \tag{2}$$

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