Journal of Constructional Steel Research 104 (2015) 37–48

Contents lists available at ScienceDirect

Journal of Constructional Steel Research

Experimental investigation and modeling of cyclic behavior of high strength steel

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ARTICLE INFO

Article history:
Received 24 November 2013
Accepted 13 September 2014
Available online 10 October 2014

Keywords:
High strength steel
Monotonic loading
Cyclic loading
Hysteretic model
Finite element analysis

ABSTRACT

With the increase in mechanical strength, the yield to tensile strength ratio becomes more close to 1 and the elongation ratio appears a significant decrease, indicating a deterioration in the ductility and seismic behavior of high strength steel (HSS). For seismic design, understanding of the ductility and cyclic behavior in material level is important to guarantee the abilities of structural steel members to endure expected inelastic deformation under severe earthquake actions. This paper presents an experimental evaluation on the uniaxial cyclic behavior of Q460C steel through 6 cyclic loading tests. The specimens were cut and machined from both steel plates and flanges of hot-rolled H-shaped steel with the nominal yield strength of 460 MPa. For the purpose of comparing to normal strength steel, additional cyclic loading test was conducted on Q345B steel. Full hysteretic loops were achieved for HSS as well as normal strength steel. Based on the observations of the test results, a simple piecewise model was developed for predicting the cyclic behavior of high strength steel, with considering the observed Bauschinger effect and cyclic strain hardening. To verify the accuracy of the proposed hysteretic model for HSS, quasi-static cyclic loading tests of Q460C steel beam-columns were simulated. The comparison between the experimental and predicted moment–curvature curves showed a good agreement, indicating a reasonable efficiency of the proposed trilinear kinematic hardening model in predicting the hysteretic behavior of HSS beam-columns.

1. Introduction

High strength steel (HSS) is defined as steel having a nominal yield strength no less than 430 MPa in the background document of Eurocode3 by the European Convention for Constructional Steelwork [1]. According to the current Chinese steel structures design code GB 50017-2003 [2], HSS refers to as steel grades no less than Q460 (with the nominal yield strength of 460 MPa). Since recent advances of technology in material science and increasing demand for high strength, HSS has been used in high-rise buildings, large span buildings and bridges in the past two decades. Compared with conventional mild carbon steel members, the application of HSS members could not only reduce member size and save building space, but also show considerable economic benefits through reducing the workloads of transportation as well as welding and shortening the time of construction. Previous researches focused on the ultimate bearing capacity, local and overall buckling behavior of HSS columns [3–7] have shown that, with the increase in steel strength, the effects of initial geometric imperfection and residual stress on the buckling behavior of steel columns become less detrimental. The experimental and numerical investigations on the ductility and buckling behavior of HSS beams [8–11] have shown that the current specifications can give a rational prediction of the flexural strength of HSS beams and the modification to the existing compactness criteria is suggested to guarantee a required rotation capacity for HSS beams. Therefore, with minor modifications, the current steel structure design standards can be used for the elastic design of HSS structures at non-seismic regions. However, for seismic design, it is not economical to restrict structures in elastic state under severe earthquake actions. Instead, it is expected that structures should be able to endure rational inelastic deformations in order to dissipate earthquake energy. Therefore, the seismic performance of HSS should be examined if it is related to the application of HSS in structural design with required earthquake-resistance. With the increase in yield strength, the yield to tensile strength ratio gets close to 1.0 and the elongation ratio decreases. Ductility requirements specified in Eurocode3 [12] and Eurocode8 [13] are in terms of the tensile to yield strength ratio ($f_u/f_y \geq 1.10$), the elongation ratio (not less than 15%) and the ultimate strain (not less than 15 times of the yield strain). According to the Chinese Code for Seismic Design of Buildings GB 50011-2010 [14], the ductility requirements of steel ($f_y/f_u \leq 0.85$, elongation ratio $\geq$ 20%) are more stringent than those of Eurocode3 due to the expectation of inelastic behavior of structural elements and connections under rare earthquake actions. For HSS, those important mechanical properties of

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http://dx.doi.org/10.1016/j.jcsr.2014.09.009

seismic design can hardly meet the requirements specified in GB 50011-2010 and Eurocode3. Currently, the application of HSS in seismic design is restricted by existing seismic design codes and constructional practice, which are established on the study of conventional steel. Therefore, better understanding of the inelastic cyclic behavior of HSS, as well as the reliable model for predicting the seismic behavior of HSS members, is important for applying HSS to seismic resistance structures.

A number of experimental observations have been made on the hysteretic behavior of metal materials subjected to uniaxial cyclic loading. Based on the observations of macroscopic and microscopic behaviors, those analytical models were generalized and proposed for predicting the cyclic stress–strain behavior of metals, which can be classified into two groups, plasticity models and continuum models [15]. The performance of the model depends on whether it can accurately describe the Bauschinger effect as well as cyclic strain hardening or softening during the entire cyclic loading history. To this end, the classical kinematic and isotropic hardening models were further refined in order to achieve a better agreement with the experimental results under cyclic loading. The multi-surface model was independently developed by Mroz [16] and Iwan [17] by combining the kinematic and isotropic hardening models in terms of a family of surfaces in stress space. In order to offer some features of simplicity, the two-surface model which consisted of an inner yield surface and an outer bounding surface was proposed by Dafalias and Popov [18,19]. Based on the bounding surface concept, more constitutive models with specific treatments for cyclic response of structural steels have been developed. Shen et al. [20–23] proposed a generalized two-surface model to include the yield plateau of mild carbon steel. This model was calibrated against mild steel under torsion–compression loading. Ucak and Tsopelas [24,25] combined a memory surface with the nonlinear kinematic hardening concept [26] to reproduce the progressive cyclic hardening or softening effect and verified the proposed model at the structural member level. Mahan et al. [27] simplified the description of yield plateau by introducing a fixed non-hardening bounding surface under a cumulative plastic strain threshold. On the other hand, Goto et al. [28], Yoshida and Uemori [29] and Jia and Kuwamura [30] improved the prediction of cyclic behavior of mild steel in large plastic strain ranges within the frame of the two-surface model. In addition, plasticity models for cyclic loadings were investigated in different ways. A hysteretic model for reinforcing steel was proposed by Dodd and Restrepo-Posada [15]. The model was established in the natural coordinate system to achieve the symmetrically cyclic stress–strain loops in tension and compression. A dissipation function in terms of an ellipse equation was developed by Amde and Mirmiran [31]. The inelastic part of the hysteretic curve can be conveniently calculated by the summation of the Ramberg–Osgood skeleton curve and their proposed dissipation function. Comprehensive literature reviews and comparisons of such models were given by Bate and Wilson [32] and Chaboche [33,34]. However, the aforementioned models were only calibrated by normal strength steels rather than HSS except that the Yoshida–Uemori model was compared with a dual-phase HSS sheet, which usually is not recognized as structural steel. Recently, Dusicka et al. [35] experimentally investigated the inelastic cyclic behavior and low-cycle fatigue life of plate steels with the nominal yield strength up to 485 MPa. The Ramberg–Osgood model was used to fit the experimental skeleton curves. The associated coefficients were obtained from data regression of the test results. Although the power function base on the Ramberg–Osgood model seems efficient to describe the nonlinear part of the cyclic skeleton curve, it is inconvenient to implement in numerical analysis in the structural level due to the implicit expression of stress in the power function. To this end, an explicit expression of stress in terms of rational function with four fitting parameters was proposed by Shi et al. [36]. Parameters of the model were calibrated by Q460D steel (nominal yield strength of 460 MPa) based on the test results of 17 coupon specimens. However, the yield points of reversed loading segments under both compressive and tensile states were defined as lying in the strain axis (0 stress line). Consequently, by this definition, the efficient of the hysteretic model in accounting for Bauschinger effect was reduced. Moreover, due to the lack of cyclic test data of HSS structural members, the verification of the hysteretic model on member level for HSS was limited in the previous researches. Therefore, the necessity of developing and calibrating a cyclic model for HSS is highlighted.

This paper is concerned with the uniaxial cyclic behavior of Q460C steel. A total of 12 tensile coupons and 6 cyclic loading specimens were tested. The coupons were machined from 11 mm and 21 mm thick plates and 14 mm thick flange of hot-rolled H-shaped section. The nominal yield strength of the coupons is 460 MPa. For the purpose of comparing to normal strength steel, two Q345B coupons were tested under cyclic loading. Based on the observations of the test results, a simple piece-wise model was developed for predicting the cyclic behavior of high strength steel, with considering the observed Bauschinger effect and cyclic strain hardening. In order to verify the accuracy of the proposed HSS hysteretic model, quasi-static cyclic loading tests of Q460 steel beam-column specimens reported in an accompanied study were simulated and compared.

2. Experimental program

2.1. Test materials

Three kinds of Q460 steel were used in this test, which are 11 mm thick Q460C steel plate produced by Nanjing Iron & Steel Co., Ltd, 21 mm thick 460C steel plate produced by Baoshan Iron & Steel Co., Ltd, and hot-rolled Q460C H-shaped steel H496 $\times$ 199 $\times$ 9 $\times$ 14 produced by Maanshan Iron & Steel Co., Ltd. The chemical compositions of the test materials are summarized in Table 1. Those parameters can meet the requirements of the Chinese code for High Strength Low Alloy Structural Steels GB/T 1591-2008 [37]. Tensile coupon tests were carried out to determine the stress–strain characteristics of Q460 steels. The cutting direction was perpendicular to the rolling direction for steel plate or parallel to the rolling direction for H-shaped steel. Fig. 1a shows the details of the machined coupon specimens.

A total of 12 tensile coupon tests were conducted in accordance with GB/T 228-2002 [38]. The average values of the tests are summarized in Table 2. In Table 2, E is the Young's modulus, f_y is the 0.2% proof stress, which is adopted as the yield strength of steel plates, f_u is the ultimate tensile stress, and A is the percentage of elongation after fracture. The tested Q460 steels show a much better ductility than the minimum limits specified in GB/T 1591-2008 [37] and EN 10025-6 [39]. Plate steel with the thickness of 11 mm shows the lowest ductility among the tested Q460C steels. Nevertheless, the elongation ratio of the 11 mm Q460 plate steel is higher than the requirements specified in Eurocode8 [13] and GB 50011-2010 [14] by 58% and 19%, respectively. And the yield to tensile strength ratio of the tested Q460C steel can satisfy the ductility requirements of Eurocode8 [13] and GB 50011-2010 [14]. Fig. 1b shows the typical stress–strain curves of Q460C plate steels

Table 1
Chemical composition of Q460C steel in percentage of weight.

Specifications	C	Si	Mn	P	S	Nb	V	Cr	Ni	Cu	Ceq
11 mm plate steel	0.07	0.24	1.62	0.011	0.002	0.05	0.048	0.04	0.02	0.05	0.35
21 mm plate steel	0.06	0.20	1.56	0.010	0.002	0.05	0.030	–	0.18	0.16	0.35
14 mm flange of H-shaped steel	0.18	0.43	1.40	0.021	0.009	0.03	0.067	0.02	0.007	0.21	0.43