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Experimental and numerical investigations of Q690D H-section columns under lateral cyclic loading

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ABSTRACT

To evaluate hysteretic performance of welded Q690D high-strength steel H-section columns, cyclic loading test and numerical simulation were carried out. Firstly, cyclic loading test was performed on two H-section Q690D HSS welded columns with nominal axial compression ratio of 0.35. The observed failure mode is elastic–plastic local buckling of component plates. Hysteretic curves and skeleton curves of the test specimens show that the tested columns have a good energy dissipation capacity. Then, a finite element model was developed to simulate the hysteretic behavior of Q690D beam-columns and verified through comparison of numerical and experimental results. It is found that residual stresses are of ignorable influence on the ultimate bearing capacity of beam-columns. With the verified finite element model, extensive parametric analyses were conducted to investigate the effects of component plate slenderness ratio, column slenderness ratio and axial force ratio on the hysteretic performance of the Q690D H-section columns. Based on the observation and analyses of the experimental and numerical hysteretic curves, a trilinear moment-curvature hysteretic model for welded Q690D steel H-section column was proposed.

1. Introduction

Application of high-strength steel (HSS) can reduce member size as well as save structural steel usage. The advantages lead to an increasing use of HSS in high-rise buildings, large space structures and long-span bridges [1]. Considering both the safety and economy, steel structures are generally designed to be able to endure certain inelastic deformation under severe earthquake. Thus, it is important to have a good understanding of the hysteretic behavior of structural members. The seismic behaviors of normal strength steel members have been extensively investigated, including the influences of geometrical parameters and loading conditions. Ballio and Castiglioni [2] characterized the cyclic behavior of steel beams based on the test and numerical simulation of 29 specimens with I-shaped sections. They discussed cumulative damage models for predicting the seismic behavior of steel members. Fadden and McCormick [3] carried out cyclic quasi-static test on 11 hollow structural section beam members. They verified that hollow structural sections could be used in seismic applications when width to thickness ratio and height to thickness ratio were carefully determined to ensure a stable plastic hinge behavior. The behavior of columns subjected to changing compression axial load was investigated by Lamarche and Tremblay [4]. The test results show that steel columns with compact I-shaped sections might be able to sustain a certain inelastic buckling without loss in load carrying capacity under earthquake actions. According to nonlinear time–history analysis of structures subjected to earthquake ground motions, columns are usually subjected to combined axial load and inelastic rotation derived from story drift. The effects of width to thickness ratio, height to thickness ratio and axial load ratio on the cyclic behavior of steel beam-columns have been evaluated. Newell and Uang [5] tested nine full-scale wide-flange columns under high axial force ratio of 35%, 55%, and 75% combined with story drift ratio up to 10%. It is concluded that the tested columns can sustain a large inelastic rotation under high axial loads. Similarly, experimental studies on box columns were carried out by Nakashima and Liu [6] and Kurata et al. [7]. It shows that the box column with compact section can sustain medium axial load (30% of the sectional yield strength) under story drift rotation up to 30% after the occurrence of local buckling. However, the prediction of crashing failure behavior by finite element analysis deviates from the experiments. This indicates the strong need for further experimental investigations. Not only in steel frames, are hollow box-section columns also widely used in steel bridge piers. However, it is typical with larger width to thickness ratio but lower in axial force ratio than those columns in buildings. Kumar and Usami [8] tested 10 hollow box piers under low axial force ratios (16.3% and 20% of the yield strength of piers) combined with cyclic lateral load. They developed a damage model to quantify the damage sustained under earthquake actions. Moreover, Aoki and Susantha [9] investigated the effect of cross-sectional aspect ratio on ultimate strength, ductility and energy dissipation capacity of rectangular-shaped steel piers. The height to

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Table 1
Mechanical properties of Q690D steel.

Specimen	f_y MPa	f_u MPa	f_y/f_u	ε_u	Δ (%)
L16-B1	780	834	0.93	0.0590	18
L16-B2	772	827	0.93	0.0546	20
L16-B3	782	837	0.93	0.0625	20
L16-B4	781	836	0.93	0.0604	19
L16-B5	780	835	0.93	0.0586	19
Mean value	779	834	0.93	0.0590	19

width ratio of 1.6 was suggested for the optimal design of member ductility and energy dissipation capacity.

As pointed out by Fukumoto [10], the increase in strength will result in the increase in yield to tensile strength ratio and the decrease in elongation ratio. Due to the differences in mechanical properties, the seismic performance of HSS members may be different from that of the conventional normal strength steel members [11]. European and American specifications for steel structures allow the use of HSS up to steel grades of S700 (700 MPa) and A514 (690 MPa) [12,13]. However, the existing design codes are established based on the experimental and analytical studies of mild carbon steels usually with nominal yield strengths from 235 MPa to 355 MPa [14,15]. As specified in Eurocode3 [16] and Eurocode8 [17], (1) tensile to yield strength ratio shall not be less than 1.10 ($f_y/f_u \le 0.91$), (2) elongation ratio shall not be less than 15% and (3) ultimate strain shall not be less than 15 times of the yield strain. With increase in yield strength, the yield to tensile strength ratio gets close to 1, meanwhile the elongation ratio decreases. Thus, for HSS, it is difficult to meet the requirements for seismic design. To this end, Wang and Li et al. [18,19] evaluated the seismic performance of welded box- and H-section beam-columns with nominal yield strength of 460 MPa. They found that Q460C steel beam-columns with cross sections of class 1 and 2 show a good capacity of energy dissipation. The class 1 cross-section cannot guarantee to provide a cyclic rotation capacity with unreduced resistance under severe earthquake. For class 2 cross-sections, only limited rotation capacity can be utilized under laterally cyclic load as the expected local buckling. However, the knowledge of the hysteretic behavior of the beam-columns with higher steel grades is limited due to lack of test data. In this paper, experimental and numerical studies on the cyclic behavior of welded H-section beam-columns with nominal yield strength of 690 MPa are presented. Two welded H-section columns were tested under cyclic loading combined with nominal axial force ratio of 0.35. Ultimate strength, ductility and energy dissipation capacity were evaluated according to the observation of the obtained lateral load–displacement hysteretic curves. Moreover, a finite element (FE) model was developed and validated through comparison between the simulation and test results. Based on the verified FE model, parametric analyses were conducted to investigate the effects of axial force ratio, component plate slenderness ratio and column slenderness ratio on the hysteretic performance. Through the generalization of the experimental and numerical results, a moment-curvature hysteretic model for Q690D welded H-section column was proposed.

Table 3
Limits of component plate slenderness for 690 MPa steel compression members.

Code	Limits	I	II	III	IV
Eurocode3	$(b-t_w)/t_f$	5.3	5.8	8.2	–
	$(h_0-2t_w)/t_w$	19.3	22.2	24.5	–
GB 50011-2010	b/t_f	5.8	6.4	7.0	7.6
	h_0/t_w	25.1	26.3	28.0	30.3

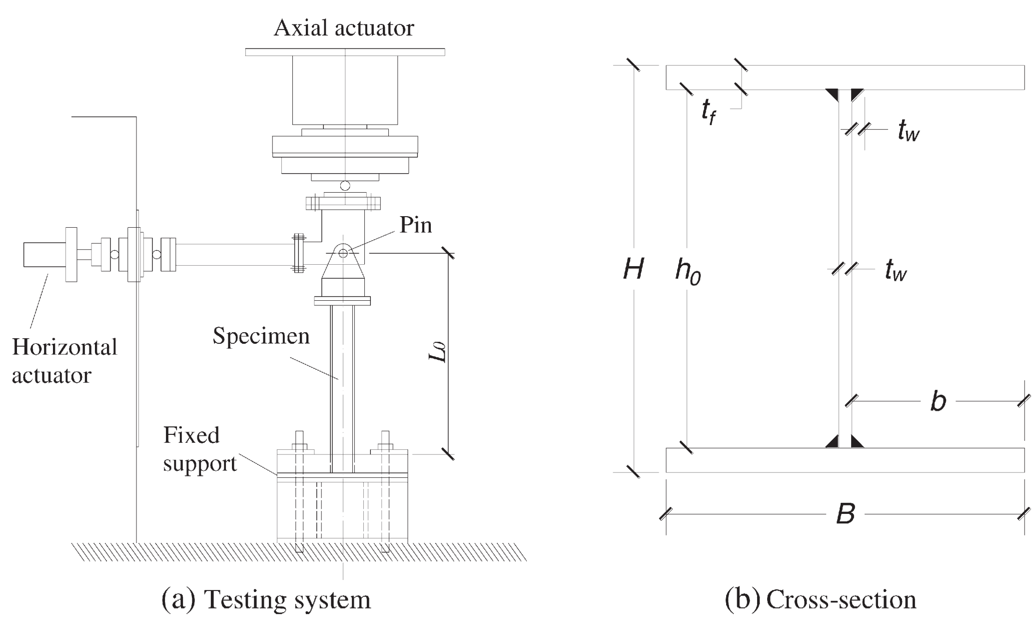

Fig. 1. Test setup.

Table 2
Dimensions of H-section specimens.

Specimens	H mm	B mm	t_w mm	t_f mm	L_0 mm	b/t_f	h_0/t_w	λ_x	n	P_y kN	M_y kN·m	M_p kN·m	θ_y%
H-1 and H-2	250	250	16	16	2505	7.3	13.6	48.3	0.35	7927	769.6	877.2	1.45