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The development of the direct strength method for welded steel members with buckling interactions

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ABSTRACT

The direct strength method (DSM) has been adopted by the NAS (2004) and AS/NZS 4600 (2005) for the design of cold-formed steel members. The method can be successfully applied to the design of welded and hot-rolled sections. This paper reviews the development of the DSM for welded steel structural members. The design strength formulae for welded section columns and beams for the DSM are provided based on the tests performed on welded H-section, C-section, circular and rectangular hollow section columns fabricated from steel plates whose nominal yield stress is 235 MPa or 315 MPa. The comparison between the design strength of welded sections predicted by the DSM and that estimated by existing specifications is provided. This paper verifies that the DSM which adopts the nominal axial strength and flexural strength in the AISC (2010) or EC3 (2004) can properly predict the ultimate strength of welded section columns and beams.

1. Introduction

In general, welded steel section compression and flexural members may normally buckle in local and overall buckling modes [1–3], whereas cold-formed steel sections may buckle in the distortional mode in addition to these modes [4]. However, longitudinally stiffened box sections used for large scale girders or columns also buckle in the distortional mode [5]. When local or distortional buckling stress is lower than overall buckling stress, the interaction between local buckling and overall buckling or between distortional buckling and overall buckling may occur and have a significant effect on the performance of structural steel sections. Since the interaction between buckling modes generally deteriorates overall member strength, it is necessary to account for the negative effect of buckling interactions in the conservative prediction of the ultimate strength of columns and beams. Therefore, the ultimate axial strength of compression members composed of thin plate elements is dependent on both the width-to-thickness ratio of plate elements and the slenderness ratio of columns [1,2]. Similarly, the ultimate moment strength of flexural members is dependent on both the width-to-thickness ratio of plate elements and the laterally unsupported length [6–8]. Because the local buckling mode has a post-buckling strength reserve, it is generally considered in design specifications through the concept of the effective width [9–14], which has been used in the design of thin-walled steel sections for over seven decades. However, the effective width method is often not sufficient for accounting for distortional buckling [15].

The direct strength method (DSM), proposed by Schafer and Pekoz [16], extended by many researchers [17–19], and adopted in 2004 as an alternative to the effective width method, has been used for the design of cold-formed steel members by NAS Supplement 1 (2004) [20] and the Australian/New Zealand Cold-Formed Steel Structures Standard AS/NZS 4600 [21]. However, since the buckling interaction in welded steel sections is less severe than in cold-formed steel sections, few studies examine the application of the DSM to welded steel sections. Kwon et al. [22] are the first to apply the DSM to the design of welded section columns based on compression test results for H- and C-section columns, and the proposed strength formulae have been calibrated to test results for welded rectangular hollow section (RHS) columns [23]. The design strength formulae for the DSM have been proposed for predicting the axial strength of longitudinally stiffened plates [5], and the method has been extended to predict the design flexural strength of welded H-section beams based on simple bending tests of H-sections [24]. Similarly, the method has been applied to concrete filled tubular columns [25]. This paper provides a brief review of the development of the DSM for welded steel section members and highlights its advantages in terms of the practical design of welded section beams and columns with buckling interaction.

2. Test sections

A detailed test program for a wide range of welded steel sections has been considered at Yeungnam University to determine the

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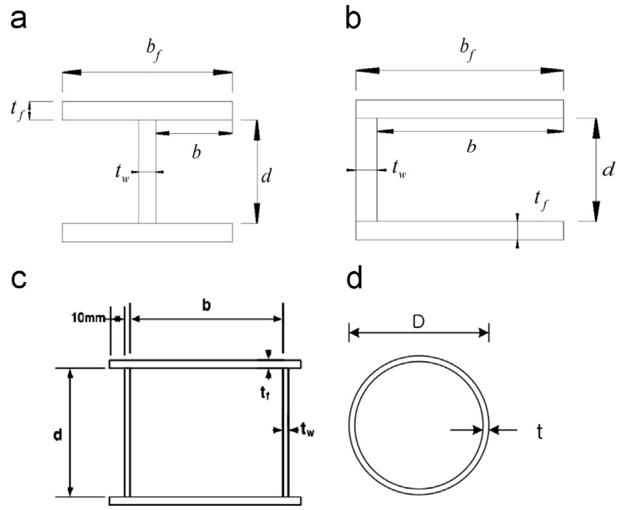

Fig. 1. Cross-sections of test specimens. (a) H-section, (b) C-section, (c) RHS and (d) CHS.

ultimate strength of sections failing in a mixed mode of local or distortional buckling and overall buckling. The geometry and dimension are described for H-section, C-section, CHS and RHS columns and stiffened plates for compression tests and H-section beams for flexural tests.

2.1. H-section, C-section, RHS and CHS columns

A series of compression tests were performed on welded H- and C-section columns of 6.0 mm thickness whose nominal yield stress and ultimate stress were 240 MPa and 400 MPa, respectively [22]. Similar tests were performed on welded RHS columns of 6.0 mm thick whose nominal yield stress and ultimate stress were 315 MPa and 490 MPa, respectively [23]. CHS columns of 3.2 mm thick whose nominal yield stress and ultimate stress were 240 MPa and 400 MPa, respectively were also tested to failure. Test specimens were fabricated by the continuous fillet welding width on both sides of the flange–web joint. The size of fillet welds was 6.0 mm based on the AISC specifications [11]. The geometry of welded H, C, and rectangular hollow sections tested are shown in Fig. 1. The limiting width-to-thickness ratio for the Class 3 cross-section is expressed as $d/t_w \le 42\sqrt{235/F_y}$ for the web and $b/t_f \le 14\sqrt{235/F_y}$ for the flange of sections based on Eurocode3 [12,13]. The width-to-thickness ratio for the web of H- and C-sections ranged from 25.0 to 91.7 and that of the flanges ranged from 7.8 to 49.5. Since these values exceed the slenderness limit value, the selected H- and C-sections are classified as Class 4 in EC3, in which the effective width must be used to account for the local buckling effect. The width-to-thickness of the web and flange of test RHS columns ranged from 26.7 to 51.7, which indicated that test sections fell in category 3 and 4 sections. The diameter-to-thickness ratio of test CHS columns ranged from 47.0 to 170.0. The limiting diameter-to-thickness ratio for the Class 3 cross-section for CHS is expressed as $D/t \le 90(\sqrt{235/F_y})^2$ in the Eurocode3 (2003) [12]. Since the diameter-to-thickness ratio for the test sections exceeds the limit value, the selected test sections are classified as Class 4 in EC3. The Eurocode3 adopts the effective width method for C- and H-section RHS columns but recommends reference to 'parts 1–6: Design of Shell Structures' for CHS columns of category 4. The AISC specifications adopted effective area to account for local buckling of CHS columns. When diameter-to-thickness ratio is $0.11E/F_y < D/t \le 0.45E/F_y$, the reduction factor Q which is the ratio of effective area to gross area of cross-sections should be determined by

$$Q = \frac{0.038E}{F_y(D/t)} + \frac{2}{3} \tag{1}$$

The dimensions of H- and C-section, RHS, and CHS columns are summarized in references [5,22–25], respectively. The slenderness ratio for H- and C-section column specimens ranged from 22.9 to 63.5, that of RHS specimens ranged from 40.3 to 67.3 and that of CHS specimens were approximately 12.0. The dimensions of test specimens were optimized to ensure that local buckling stress was lower than overall buckling stress and thus that the interaction between local and overall buckling occurred before the ultimate load was reached. The cross-sections tested were optimized by using the finite strip analysis program BAP [26] repeatedly. This program can account for the inelasticity of the material and the residual stress distribution for inelastic buckling analysis. The width-to-thickness ratio for the web and flange of test sections was selected such that the elastic local buckling stress of the section was low and a significant post-local-buckling strength reserve was displayed before the ultimate load.

2.2. H-section beams

Simple bending tests were performed by Kwon and Seo [24] on welded H-section beams fabricated by gas welding from SM490 structural steel plates of 6.0 mm thickness whose nominal yield stress and ultimate stress are 315 MPa and 490 MPa, respectively. The geometry of test sections was proportioned to undergo local buckling and fail in a mixed mode of local and lateral–torsional buckling. Cross-section of test beams is shown in Fig. 1(a). Compact and non-compact limits for flexural members in AISC specifications (2010) [11] are $0.38\sqrt{E/F_y}$ and $0.95\sqrt{k_cE/0.7F_y}$ for the flange and $3.76\sqrt{E/F_y}$ and $5.70\sqrt{E/F_y}$ for the web of welded H-sections. The yield limits of the width-to-thickness ratio (b/t) in the KHBDS (2010) [27], which are equivalent to the non-compact limits in AISC specifications, are 11.2 for the flange and 131.2 for the web. The width-to-thickness ratio of the flanges of test sections ranged from 12.5 to 33.3, and that for the web ranged from 66.7 to 100.0. Test sections were composed of compact and non-compact stiffened elements for the web and non-compact or slender and unstiffened elements for the flange based on the AISC specifications (2010). Based on EC3 [12], flanges of test sections are categorized as class 3 and 4, and webs, class 2 and 3. In terms of the width-to-thickness ratio, it was assumed that test sections might undergo mainly flange local buckling and lateral–torsional buckling simultaneously during testing.

2.3. Stiffened plates

The concentric compression tests by Kwon and Park [5] were conducted on longitudinally stiffened steel plates of 4.0 mm in thickness whose nominal yield stress and ultimate stress are 235 MPa and 400 MPa, respectively. The stiffened plates with longitudinal stiffeners were fabricated by continuous fillet welding to an effective width on both sides of the stiffener-plate joint. The size of the fillet weld was determined as 4.0 mm based on the AISC specifications [11]. The test sections were proportioned to fail in the distortional buckling mode or mixed mode of local buckling and distortional buckling. Fig. 2 shows the geometry of the stiffened plate sections where T-shape stiffeners were attached

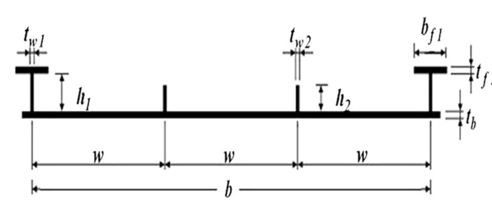

Fig. 2. Cross-section geometry of the stiffened plate.