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Prediction of the flexural strengths of welded H-sections with local buckling

Young Bong Kwon*, Gun Ho Seo

Department of Civil and Environmental Engineering, Yeungnam University, Gyongsan, Korea

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ABSTRACT

This paper describes the flexural strength of welded sections based on a series of flexural tests performed on H-sections fabricated from steel plates of thickness 6.0 mm with nominal yield stress of 315.0 MPa. Thin-walled flexural members undergo local, lateral-torsional or their interactive buckling according to the section geometries and lateral boundary conditions. Flexural members with the flanges or the web of large width-to-thickness ratios may undergo local buckling before lateral-torsional buckling and their interaction before the final collapse of the section. The local buckling has a negative effect on the flexural strength based on the lateral-torsional buckling. This phenomenon should be considered in the estimation of the nominal flexural strength of thin-walled flexural members. Welded H-section beams composed of the flanges and the web with various width-to-thickness ratios were tested to failure. The initial imperfections in local and lateral buckling mode, and residual stresses were included in the FE analyses. Simple design flexural strength formulas for the direct strength method (DSM) were proposed based on the test and FE results of welded sections to account for interaction between local and lateral-torsional buckling. The design strength curves were compared with the AISC specifications (2005), Eurocode3 (2003) and test results. The adequacy of the strength curve for the DSM was confirmed. A set of conclusions on the flexural strength and structural behavior of thin-walled welded H-sections was drawn from the experimental studies.

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1. Introduction

Flexural members composed of thin-walled plate elements undergo local, lateral-torsional or their interactive buckling according to width-to-thickness ratios of plate elements and the unsupported length. Flexural members composed of compact elements where the width-to-thickness ratios are smaller than a certain limit undergo lateral-torsion buckling or yielding of material without local buckling. Flexural members with large width-to-thickness ratio in the flanges or the web may undergo local buckling before lateral-torsional buckling and interaction between them may occur before final failure [1,2]. Though there is a significant post-buckling strength in a local buckling mode, the local buckling has a negative effect on the strength of flexural members based on the overall lateral-torsional buckling [3]. This phenomenon should be considered to predict the ultimate flexural strength of thin-walled flexural members accurately. However, in the case that the lateral rigidity is enough not to allow later-torsional buckling in the unsupported length, local buckling strength governs the moment capacity of flexural members. The effects of local buckling in the flanges or the web on the moment capacity of the flexural members were studied by many researchers [1,4–6]. To account for local buckling and post-local-buckling strength reserve in the design strength, AISC specifications (2005) [7] and AASHTO specifications (2007) [8] takes the minimum value between flange local buckling strength and lateral-torsional buckling strength as nominal moment capacity and uses load reduction factor to account for web local buckling. NAS (2004) [9] and Eurocode 3 (2003) [10,11] adopt effective width method to account for flange and web local buckling. KHBDS(2010) [12] has basic design strength curves in separate to account for local buckling of stiffened plate and unstiffened plate, and lateral-torsional buckling strength and takes the minimum value as the nominal flexural strength of the sections.

The direct strength method (DSM) has been developed by Schafer and Pekoz [13] and studied further by many researchers [14–16]. The method has been developed to overcome the weak points of the effective width method (EWM), which has been used in thin-walled steel section design for over 60 years. These weak points are the complication in accurate computation of effective width and the difficulty in consideration of the elements interacting in isolation. Also, as cold-formed steel sections become more complex with additional edge and intermediate stiffeners, calculations of effective width can become very complex. In 2004, the direct strength method was adopted formally as an alternative to the effective width method, which has been used for thin section design by NAS Supplement 1 (AISI, 2004) [9] and the Australian/New Zealand Cold-Formed Steel Structures Standard AS/NZS 4600 [17]. The application of the DSM was studied for the welded H-section and C-section columns with interaction between

* Corresponding author. Tel.: +82 11 802 2418; fax: +82 53 810 4622.
E-mail address: ybkwon@ynu.ac.kr (Y.B. Kwon).

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local and overall buckling [18] and for longitudinally stiffened plates undergoing interaction between local and distortional buckling [19] recently.

This paper aims to develop the design flexural strength formulae for welded sections. The application of the DSM to welded section flexural members was studied experimentally and theoretically. A series of flexural tests was performed on welded H-sections fabricated from structural steel plates of thickness 6.0 mm with nominal yield stress 315.0 MPa to develop the flexural strength formulae for the welded steel sections undergoing interaction between local and lateral-torsional buckling. Nonlinear FE analyses of the sections tested have also been conducted to compare their results with the test results. The strength formulae for the DSM have been proposed and compared with the current design specifications. The strength curves proposed have been proven accurate and efficient to predict the nominal flexural strength of welded sections when the local buckling and the lateral-torsional buckling occur simultaneously or nearly simultaneously.

2. Test sections and material properties

2.1. Material properties

The structural steel grade of the test sections chosen was SM490 (KSD 3515) [20]. The minimum specified yield and ultimate tensile stresses were 315.0 MPa and 490.0 MPa, respectively. Tensile coupon tests were conducted for flat coupons cut from the fabricated sections. All coupons were tested in a 250 kN capacity UTM (Schmazu AUTOGRAPH AG 250kNG) at a displacement rate 0.1 mm/min. The yield and ultimate tensile stresses of the coupons cut from near the flange-web joint lines were slightly higher than those of the coupons cut from the flange tip and web center of the specimens. Average yield and tensile stresses obtained from the tensile coupon tests were 413.0 MPa and 555.0 MPa, which were 31.1% and 13.3% higher than the specified values, respectively. Average elongation measured was approximately 25.0%. The elastic modulus measured was 1.95×10^5 which was slightly lower than the nominal value of 2.05×10^5 MPa.

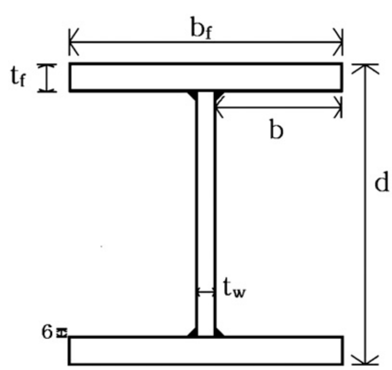

Fig. 1. Cross section geometry of test section.

2.2. Section geometries

The test H-sections were fabricated by gas welding from SM490 structural steel plates of thickness 6.0 mm. The geometries of the test specimens were determined using the buckling analysis program BAP [21], which assumes pinned end boundary conditions. The program BAP can account for the inelasticity of the material and the residual stress distribution for inelastic buckling analysis. The test section geometries determined were proportioned to undergo local buckling and to fail in a mixed mode between local and lateral-torsional buckling.

Bearing stiffeners of 10 mm in thickness were attached by continuous fillet welding at both supporting ends and at the one-third points of overall length of the sections where concentrated knife edge loads were applied to prevent web crippling or yielding and flange local deformation. Additional horizontal stiffeners were attached at both sides of the web outside of pure bending zone in order to restrain web local buckling due to shear and bending moments near both end supports before local buckling or lateral-torsional buckling at pure bending zone due to pure bending moment. Test section geometry is shown in Fig. 1.

Detail dimensions of test specimens are summarized in Table 1. Compact and non-compact limits for flexural members in AISC specifications (2005) [7] are $0.38(E/F_y)^{1/2}$ and $0.95(k_cE/0.7F_y)^{1/2}$ for the flange, and $3.76(E/F_y)^{1/2}$ and $5.70(E/F_y)^{1/2}$ for the web of H-sections, respectively. The yield limits of width-to-thickness ratio (b/t) provided by KHBDS (2010) [12] which are equivalent to non-compact limit in AISC specifications are 11.2 for flange and 131.2 for web, respectively. As shown in Table 1, the width-to-thickness ratio of flanges ranged from 12.5 to 33.3 and those of web ranged from 66.7 to 100.0. Test sections were composed of compact and non-compact stiffened elements for webs and non-compact or slender unstiffened elements for flanges according to the AISC specifications (2005). Flanges of test sections were categorized as class 3 and class 4, and webs were class 2 and class 3 according to EC3 [10]. Referring to the width-to-thickness ratios, it was supposed that test sections might undergo mainly flange local buckling during testing.

Table 1
Detail dimensions of test sections.

Specimens	b_f (mm)	t_f (mm)	d (mm)	t_w (mm)	b_f/t_f	h/t_w	l (mm)	I_x (mm^4)
H150-400	150.0	6.0	412.0	6.0	12.5	66.7	3000.0	106,181,600.0
H150-500	150.0	6.0	512.0	6.0	12.5	83.3	3000.0	177,721,600.0
H150-600	150.0	6.0	612.0	6.0	12.5	100.0	3000.0	273,261,600.0
H250-400	250.0	6.0	412.0	6.0	20.9	66.7	3000.0	155,636,000.0
H250-500	250.0	6.0	512.0	6.0	20.9	83.3	3000.0	254,536,000.0
H250-600	250.0	6.0	612.0	6.0	20.9	100.0	3000.0	383,436,000.0
H250-400	250.0	6.0	412.0	6.0	20.9	66.7	5000.0	155,636,000.0
H250-500	250.0	6.0	512.0	6.0	20.9	83.3	5000.0	254,536,000.0
H250-600	250.0	6.0	612.0	6.0	20.9	100.0	5000.0	383,436,000.0
H350-400	350.0	6.0	412.0	6.0	29.2	66.7	10,000.0	205,090,400.0
H350-500	350.0	6.0	512.0	6.0	29.2	83.3	10,000.0	331,350,400.0
H350-600	350.0	6.0	612.0	6.0	29.2	100.0	10,000.0	493,610,400.0
H400-400	400.0	6.0	412.0	6.0	33.3	66.7	10,000.0	229,817,600.0
H400-500	400.0	6.0	512.0	6.0	33.3	83.3	10,000.0	369,757,600.0
H400-600	400.0	6.0	612.0	6.0	33.3	100.0	10,000.0	548,697,600.0

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