Journal of Constructional Steel Research 95 (2014) 32–43

Contents lists available at ScienceDirect

Journal of Constructional Steel Research

Role of local slenderness in the rotation capacity of structural steel members

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ARTICLE INFO

Article history:
Received 11 July 2013
Accepted 19 November 2013
Available online 25 December 2013

Keywords:
Local slenderness
Member classification
Strain capacity
Rotational capacity
Plastic hinge rotation

ABSTRACT

The objective of this paper is to demonstrate how element (e.g., flange) local slenderness may be used to predict element strain capacity, and in turn, the element strain capacity may be used to predict member rotational capacity in structural steel members. Member plastic hinge rotation capacity has an important role in the design of steel structures, and while implicit understanding of the rotation capacity has sufficed in the past, as inelastic direct analysis methods are adopted in conventional as well as seismic design more explicit treatments are needed. Accordingly, a comprehensive series of material and geometric shell finite element collapse analyses are performed in ABAQUS on component elements (plates). The finite element analysis confirms the hypothesis that local slenderness of an element is intimately connected to the element's strain capacity. Utilizing element strain capacity to determine member rotational ductility demonstrates the importance of additional factors, such as depth-to-length and shape factor of the member in predicting the rotational capacity. The proposed method assumes Euler–Bernoulli beam theory, ignores interaction between local and lateral–torsional buckling, and presumes the flange (not the web) controls the section strain capacity. The analyses are compared to existing code provisions for both conventional and seismic design and recommendations for potential improvements are made.

1. Introduction

In the design of hot-rolled (structural) steel structures, classification of structural members for local buckling is a common approach in most current design codes such as AISC-360 and Eurocode 3 [1,2]. Classifications are generally considered to connect member strength or ductility capacity to element characteristics such as width-to-thickness ratio and boundary conditions (stiffened or unstiffened elements). Both of these element characteristics can be interpreted as member local slenderness by considering proper plate buckling coefficients [3]. In Chapter B of the AISC specification, sections are classified as containing compact, noncompact, and/or slender elements [1]. For each classification, a different design method or provision is presented to account for element slenderness in the determination of the member strength. These classifications are considered for members subjected to axial compression, flexure, or combined flexure and axial compression (i.e., beam-columns).

On the other hand, another section classification scheme is set forth in the AISC Seismic Provisions which addresses the ductility capacity of the section in terms of axial or rotational ductility [4]. In this classification, the sections are classified as “highly ductile” or “moderately ductile” members. Seismic Force Resisting Systems (SFRS) determine the ductility demands of the members considered to provide ductility for the system. Generally, a more ductile SFRS would impose more ductility demands on the members and therefore members should fulfill the requirement of “highly ductile” members.

Appendix 1 of the AISC specification lets designers use inelastic methods in analysis and design of non-seismic controlled structures [1]. As a requirement for the use of the inelastic analysis methods, the structural members must have a certain amount of ductility at the point of plastic hinges. Although ductility demands at the plastic hinges could be determined precisely by means of plastic analyses, instead the code requires the engineer to provide a minimum required rotation capacity ($R_{cap} = 3$) for the section, where R_{cap} is a dimensionless parameter used to show the rotation ductility of the section (see Section 4). This is established by using “compact” cross-sectional elements along with some additional modifications, and providing more closely spaced lateral bracing. Thus, the relation between cross-sectional or member characteristics and the ductility capacity is not explicitly discussed.

In seismic analysis, or the design and rehabilitation of structures, determination of the member ductility as a function of the cross-section characteristics is essential. This information is needed for both modeling parameters and acceptance criteria, as discussed in FEMA-356 and ASCE-41-06 [5,6]. Currently, both the modeling parameters and acceptance criteria for plastic deformations are connected to either

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http://dx.doi.org/10.1016/j.jcsr.2013.11.016

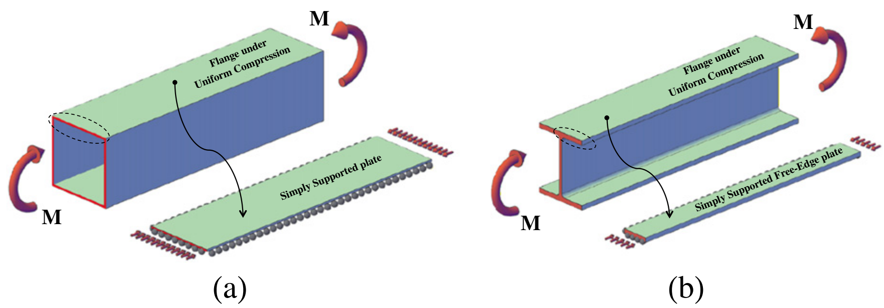


Fig. 1. Structural members under bending moment (M): (a) Box-shaped section; (b) I-shaped section.

beam flange width-to-thickness or web depth-to-thickness ratios and a linear interpolation is adopted for values in-between.

Almost all current design codes consider a stepwise approach in classification of sections that may result in non-optimal design of steel members for both strength and ductility demands. While an inherent continuous change in structural capacity in terms of strength or ductility is anticipated by changing the member (or element) slenderness, design codes provide essentially lower-bound solutions over specific classification regimes, and ignore the reserve capacities between the classification limits. As a general motivation, improving current code-based approaches and making cross-section and member characteristics more explicit in the determination of member capacity (strength or ductility) would be potentially beneficial and could culminate in more realistic and optimized steel structures.

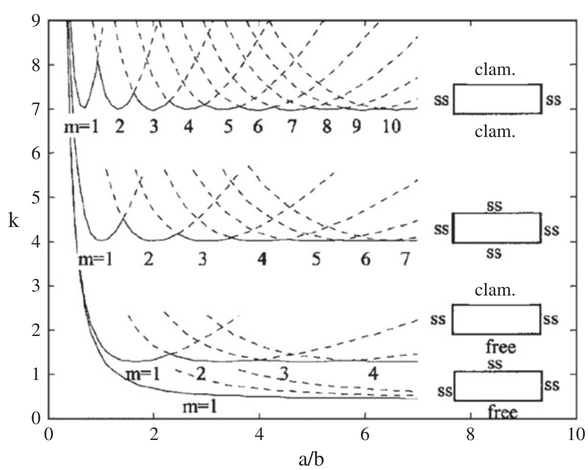
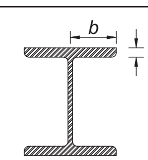
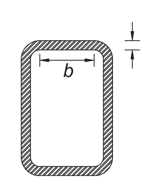


Fig. 2. Plate buckling coefficient, k, as a function of normalized plate length (a/b) for different boundary conditions, m = number of buckled half-waves along the length of the plate [16]. (ss: simply supported; clam.: clamped or fixed support; free: free edge or no support).

Another related issue on the local buckling limits of the current code is that the interaction between the elements, such as flanges and web, are not considered explicitly in all cases. A detailed study on element interaction showed that the code limitations could be modified to consider the interaction between the section elements more precisely [3]. In Kemp's foundational work, e.g. [14], element web–flange interaction is included in a plate-spring model that aims to predict inelastic capacity of I-shaped steel beam-column members with local and lateral–torsional buckling interaction. Kato's complementary efforts, e.g. [15], focus on rotation capacity of H-shaped members based on the inelastic strain capacity of the compression flange indirectly calculated based on the flange local buckling critical stress.

In recent years, a new deformation-based design approach termed the Continuous Strength Method (CSM) has been proposed [7] and adopted in AISC Design Guide 27 [8] to determine the resistance of compact and non-compact stainless steel structural members based on the deformation capacity of cross-section elements. Recently, CSM has

Table 1
AISC width-to-thickness ratio and back calculated plate buckling coefficients (k).

Description [a,b,c]	Index[a,b]	Width-to-thickness ratio [a,b] b/t	Buckling [c] coefficient k	Limiting slenderness λ_l [d]	Example [a]
Flexure in flanges of rolled I-shaped sections and channels	λ_r	$1.0\sqrt{E/F_y}$	1.1	1.0	
	λ_p	$0.38\sqrt{E/F_y}$	1.1	0.38	
	λ_{md}	$0.38\sqrt{E/F_y}$	1.1	0.38	
	λ_{hd}	$0.30\sqrt{E/F_y}$	1.1	0.30	
Uniform compression in flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections; outstanding legs of pairs of angles in continuous contact and flanges of channels	λ_r	$0.56\sqrt{E/F_y}$	0.70	0.70	
	λ_p	–	–	–	
	λ_{md}	$0.38\sqrt{E/F_y}$	0.70	0.48	
	λ_{hd}	$0.30\sqrt{E/F_y}$	0.70	0.38	
Uniform compression in flanges of rectangular box and hollow structural sections of uniform thickness subject to bending or compression; flange cover plates and diaphragm plates between lines of fasteners or welds. (Applicable to columns in SMF systems and box sections used as beams or columns)	λ_r	$1.4\sqrt{E/F_y}$	4.43	0.70	
	λ_p	$1.12\sqrt{E/F_y}$	4.43	0.56	
	λ_{md}	$1.12\sqrt{E/F_y}$	4.43	0.56	
	λ_{hd}	$0.60\sqrt{E/F_y}$	4.43	0.30	

[a] AISC-360-10 [1].
[b] AISC-341-10 [4].
[c] see [3].
[d] ν is assumed to be 0.3 in calculating λ.