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Cold-formed steel member bending stiffness prediction

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ABSTRACT

The objective of this paper is to provide a simplified method for predicting the bending stiffness of thin-walled cold-formed steel members subject to elastic (or inelastic) local or distortional buckling. In thin-walled cold-formed steel members for serviceability and consideration of deformations (second-order effects) it is important to account for the loss of cross-section bending stiffness due to the use of slender elements and/or yielding. Although existing design specifications provide some guidance on how to predict the stiffness, limited information is available for cross-sections subject to distortional buckling or undergoing inelastic local and/or distortional buckling. Existing tests and shell finite element analysis on cold-formed steel beams failing in local or distortional buckling are used as the basis for the study conducted herein. Stiffness of the conducted tests and finite element models is compared with predictions based on the Effective Width Method and Direct Strength Method. Relationships between local and distortional cross-section slenderness and the observed and predicted secant stiffness up to the peak load are examined. New design expressions to predict cold-formed steel member bending stiffness depending on cross-section slenderness are presented. The new methods are proposed for adoption in cold-formed steel specifications.

1. Introduction

Cold-formed steel (CFS) is a popular building material enjoying a wide and growing base of applications in civil structures. Current design, essentially the world over, has focused on member-level strength prediction; which, for example in beams, is complicated by the presence of material yielding, local buckling, distortional buckling, lateral–torsional buckling, and combinations thereof. Significantly less attention has been given to the issue of stiffness; however, design specifications (e.g., AISI-S100 [1]) do provide methods for approximating the reduced stiffness due to local buckling — typically using variations of the Effective Width Method (EWM). In addition, since 2007, AISI-S100 Appendix 1 [2] has provided a Direct Strength Method (DSM) approach for predicting the stiffness in local or distortional buckling.

The most common CFS beams are lipped channel and zee sections. For structural applications in North America typical thickness ranges from 1 to 3 mm, depths from 100 to 400 mm, and material yield strengths from 200 to 400 MPa. The resulting combinations of cross-section slenderness create beams that may exhibit behavior from elastic local and/or distortional buckling to significant inelastic reserve and yielding. Stiffness of thin-walled members is an especially important consideration. As members are optimized for strength, the resulting stiffness often controls design, particularly for beams. Stiffness is also important for adequately considering second-order effects (P-δ and P-Δ) in structures. Thus, having an accurate, but straightforward, approach to stiffness prediction is needed in modern CFS design.

The objective of this paper is to present the exploration of, and prediction for, the bending stiffness of CFS members. Existing tests and shell finite element analysis models on CFS lipped channel (C) and lipped zee (Z) beams in local or distortional buckling are used as the basis for this study. Existing data is processed and the realized stiffness is compared with predictions based on EWM and DSM. In addition, based on the data a new simplified approach in the spirit of the DSM approach (i.e. as a function of cross-section slenderness) is provided, and recommended for design.

2. Moment–rotation data for local and distortional buckling of CFS beams

The bending stiffness of a beam may readily be determined from the moment–rotation (or more directly moment–curvature) response. CFS beams exhibit different reductions in elastic stiffness depending on the geometry, material behavior, and the dominant buckling mode and its associated deformations. Cross-section slenderness, typically realized as $(M_y/M_{cr})^{0.5}$ where M_y is the yield moment and M_{cr} is the buckling moment (e.g. for local or distortional buckling), is generally understood to be a useful parameter for understanding the extent of reductions in the elastic stiffness. Here we focus on existing experiments and shell finite element analysis results on CFS beams in local or

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distortional buckling to characterize the bending stiffness reduction for CFS beams.

A series of experiments on CFS beams in local or distortional buckling were conducted by Yu and Schafer [3–5] and are utilized herein. CFS beams composed of back-to-back paired C and Z sections were tested in four point bending. The most important parameters in the analysis of CFS member structural behavior are the cross section properties and the failure mode. Therefore, the experimental work was composed of two test series carried out on industry-standard C and Z CFS sections varying from 92 to 305 mm deep and from 1.09 to 2.46 mm thick [6]. The testing setup was carefully designed in the first series of tests to allow the local buckling failure to form while restricting distortional and lateral–torsional buckling (Fig. 1(a)). A metal panel was through-fastened with a specific detail (pairs of fasteners through the flanges) to the top (compression) flanges in the moment span. As a result, the compression flanges were prevented from rotation, and only local buckling failure was allowed. In the second series the corrugated panel remained in the shear spans only and was removed in the constant moment region so that distortional buckling could occur (Fig. 1(b)) in the tests.

Members failing in distortional buckling typically exhibited lower capacities and a slight decrease in stiffness prior to the peak strength, as shown in Fig. 2. Greater nonlinear behavior was observed before failure in the distortional buckling tests when compared with the local buckling tests, while observable deformations in the web occured in the local buckling tests.

Shiferraw and Schafer [7] used the experiments of Yu and Schafer [3,4] to develop and validate a nonlinear collapse shell finite element (FE) model focusing on local and distortional buckling limit states in typical C and Z cold-formed steel sections. Their ABAQUS models employed S9R5 shell finite elements, idealized σ–ε curves with isotropic hardening, and idealized boundary conditions [8]. The goal of these analyses was not to recreate the tests but rather to provide an idealized model that could consistently provide local and distortional buckling failure modes in a computationally efficient manner (Fig. 3). The selected model includes only the central 1.63 m constant moment region from the tests and employs special boundary conditions at the ends and along the flanges. Loading was prescribed as a rotation about the centroid at the member ends, which were not allowed to distort. Centerline dimensions of 17 cross-sections (tests having $M_{test} > 0.95M_y$) from Yu and Schafer [3,4] models were selected. From these centerline dimensions the thickness was varied from 1.37 to 3.42 mm, resulting in 187 different models. The length of the models was determined as the minimum required to generate local or distortional buckling limit states independent of boundary conditions. Comparison against existing tests showed an overall test-to-predicted ratio of 1.06 on average with a standard deviation of 0.06 [7] thus validating the selected modeling protocols. The modeling focused on CFS sections that can develop inelastic reserve; i.e., sections with a peak bending capacity greater than the moment at first yield (Fig. 4).

To facilitate the studies herein, the maximum moment (load) level is termed the 'peak point'. And, pre-peak points are defined depending on this peak point. The raw data from the tests and shell FE models were down-sampled to 10 pre-peak points, each one in increments of 10% of the displacement at peak moment [9].

a
b

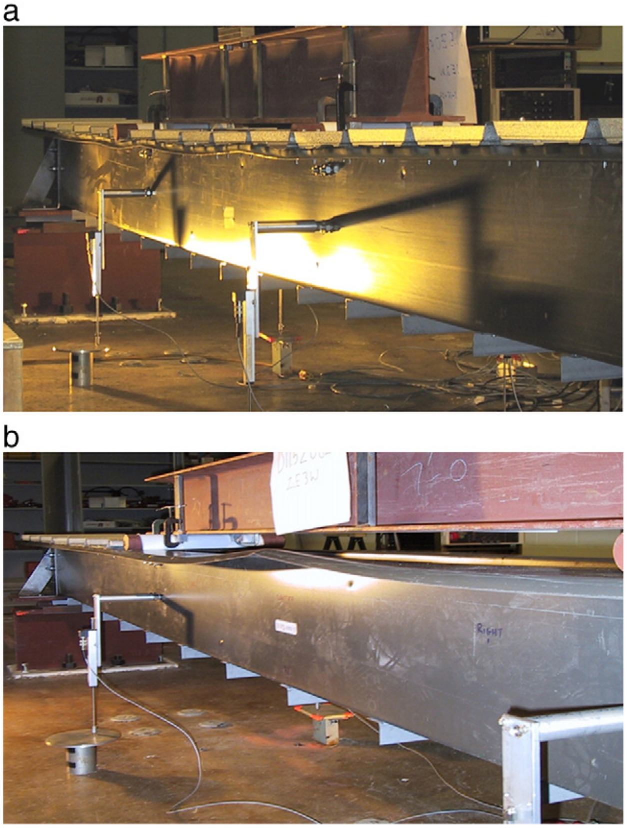

Fig. 1. Tests at failure [3,4]. (a) Local buckling. (b) Distortional buckling.

3. Cold-formed steel member stiffness

For strength, comparisons between the Effective Width Method (EWM) and the Direct Strength Method (DSM) may be found elsewhere (see, e.g., [10,11]). However, little in the way of validation or comparison has been provided for the stiffness predictions of the two methods.

3.1. Prediction based on the effective width method

Stiffness prediction by EWM [12] (as implemented in AISI-S100 [13]) may be understood as follows. The effective moment of inertia ($I_{e\text{-}EWM}$) is calculated from appropriate summation of the effective widths of all the elements:

$$I_{e-EWM} = \sum_{elements} y_e^2 b_e t \tag{1}$$

where y_e is the distance from the effective neutral axis to the effective centroid of the element, t is the thickness, and b_e is the effective width of a given element, determined from:

$$b_e = \begin{cases} b\left(1-0.22\sqrt{f_{cr}/f}\right)\sqrt{f_{cr}/f} & \text{if } \sqrt{f/f_{cr}} > 0.673 \\ b & \text{if } \sqrt{f/f_{cr}} \le 0.673 \end{cases} \tag{2}$$

where b is the gross element width, f_{cr} is the elastic local buckling stress of the element (see Chapter B of AISI-S100) and f is the applied stress at which the effective moment of inertia is to be determined. Note, $f = M/S_g$, where M is the applied moment at which the effective moment of inertia is to be determined and S_g is the gross section modulus. An interesting feature of EWM for stiffness determination is that the location in the cross-section is significant for determining the reduced stiffness, i.e., large effective width reductions in the flange result in large reductions in $I_{e\text{-}EWM}$ while large reductions in the web do not. In this study, CFS [14], a comprehensive general-purpose cold-formed steel component design tool, is used to calculate effective section properties.

3.2. Prediction based on the Direct Strength Method

Stiffness prediction by the Direct Strength Method [15] (as implemented in Appendix 1 of AISI-S100-12) assumes that the stiffness reduction follows the same trends as the strength reduction