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Characterization of in-plane backbone response of cold-formed steel beams

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ABSTRACT

Thin-walled cold-formed steel beams are investigated with existing experimental data and shell finite element simulations to characterize their in-plane moment-rotation behavior, up to and past peak strength, in local or distortional failure modes. Although ultimate strength prediction of cold-formed steel members is generally well addressed in design codes, pre- and post-peak member stiffness is only partially addressed; while member ductility and post-peak moment-rotation response suffers from a lack of any clear guidance. Without fundamental information on cold-formed steel moment-rotation/curvature response, i.e. the backbone curve, system modeling for cold-formed steel structures to collapse remains severely hampered. Existing data on cold-formed steel beams are used as the basis for the study conducted herein. Simplified moment-rotation models, motivated from ASCE 41 characterizations, defined with pre-peak flexural rigidity degradation, post-peak plateau and strength drop are explored by equating the area under the backbone moment-rotation response (energy) between the available data and simplified models. In-plane response of cold-formed steel beams is parametrized with new design expressions depending on local and distortional cross-section slenderness. This research provides work for potential incorporation into design standards such as ASCE 41 and AISI S100. Out-of-plane response of cold-formed steel beams, including lateral-torsional buckling, remains as needed future work.

1. Introduction

Collapse analysis of a building comprised of load bearing cold-formed steel framing, whether for static loads, wind loads, progressive collapse, or seismic design is predicated on knowledge of the nonlinear response of the components and connections that make up the building. After one member yields determination of the force or moment redistribution in a cold-formed steel (CFS) building system may not be accurately completed with current knowledge, requiring current design to ignore system effects. Given that CFS cross-sections are typically locally slender they have a potentially complex moment-rotation response.

Potential failure mechanisms of a CFS beam include yielding, local buckling, distortional buckling, lateral-torsional buckling, and combinations thereof. Here we focus only on in-plane, or laterally-braced behavior, thus lateral-torsional buckling is excluded. For modeling CFS structures to collapse, particularly under dynamic (seismic) loads, no current method provides guidance on member ductility and moment-rotation response for members. This in part leads to overly conservative design philosophies; for example, in CFS framed buildings all nonlinearity is assumed to be concentrated in pre-tested connections at the shear walls and any contribution from the steel framing is ignored.

This research attempts to provide needed information on the stiffness and ductility of CFS beams and provides a method for parameterizing the beam response depending on cross-section slenderness. CFS beams subjected to cross-section deformations resulting from both local and distortional buckling are examined. The paper, in Section 2, begins by introducing existing experimental data and shell finite element models on cold-formed steel beams. This is followed in Section 3 by an introduction to simplified multi-linear models for moment-rotation response, motivated primarily from ASCE 41. Section 4 summarizes recent work of the authors characterizing the pre-peak response (strength and stiffness) as a function of cross-section slenderness. Section 5 provides detailed results for characterizing the post-peak moment-rotation response as a function of cross-section slenderness in either local or distortional buckling and examines the accuracy of the developed expressions. Section 6 discusses the developed method and highlights the ability of the expressions to capture CFS beam behavior ranging from slender, with little post-peak response, to stocky with considerable inelastic yielding and ductility. This discussion is followed by conclusions and acknowledgments.

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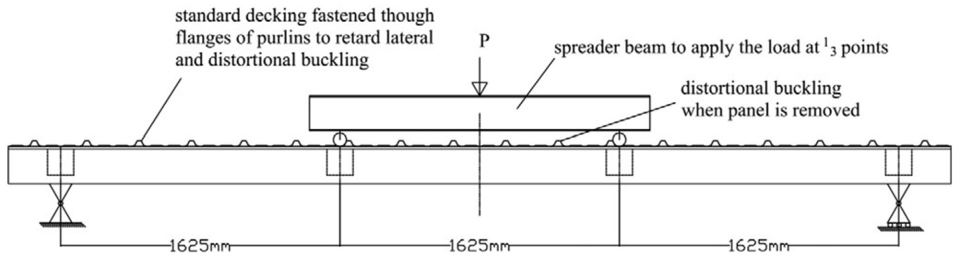

Fig. 1. Elevation view of overall test arrangement for four point bending test.

2. Cold-formed steel beam behavior – existing data and design methods

For the design of typical lipped channel (C) and lipped zee (Z) CFS beams, local and distortional cross-section buckling modes must be considered in addition to lateral-torsional buckling. Even for laterally-braced beams the in-plane behavior still must consider the potential interaction between yielding, local, and distortional buckling. Here we focus on existing experiments [3,4] and shell finite element (FE) analysis [5] of laterally-braced beams failing in elastic and inelastic local and distortional buckling.

The experiments on local and distortional buckling of CFS beams by Yu and Schafer [3,4] are utilized herein. Paired CFS beams composed of industry standard C and Z specimens varying from 92 to 305 mm deep and from 1.09 to 2.46 mm thick were tested in 4 point bending. In the first series of tests the setup, as shown in Fig. 1, was designed to allow local buckling failures while restricting distortional buckling through careful attachment of a panel to the compression flange. In the second series, the distortional buckling tests, the panel remained in the shear spans only and no restraint was provided to the compression flange of the specimens in the moment region. In these second series of tests distortional buckling failures occurred. Lateral-torsional buckling and twist were restricted in the tests by pairing the specimens at supports and load application points. In total, 24 of the local buckling tests and 22 of the distortional buckling tests from Yu and Schafer [3,4] were employed in this study for characterization of the in-plane beam response.

Shifferaw and Schafer [5] used the experiments of Yu and Schafer [3,4] to develop and validate an ABAQUS [6] nonlinear collapse shell finite element (FE) model focusing on inelastic local and distortional buckling limit states in typical C and Z CFS sections. The goal of their 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. The developed model includes only the central 1.63 m constant moment region from the tests and employs special boundary conditions at the ends (no end section distortion and warping fixity) and along the flanges (nodal point-braced at mid-width of the compression flange for local, unbraced for distortional). Centerline dimensions from seventeen cross-sections from Yu and Schafer [3,4] were selected. From these centerline dimensions the thickness was varied from 1.37 to 3.42 mm, resulting in 187 different models.

The moment capacity of beams is known to be sensitive to the cross-section slenderness (λ) which is defined as the following for local (subscript l) and distortional (subscript d) slenderness:

$$\lambda_l = \sqrt{\frac{M_y}{M_{crl}}} \tag{1}$$

$$\lambda_d = \sqrt{\frac{M_y}{M_{crd}}} \tag{2}$$

where M_y is the elastic yield moment, and M_{cr} is the elastic critical buckling moment either in local ($_l$) or distortional ($_d$) buckling. Peak strength observed in the tests and FE models correlates well with cross-section slenderness and the independently determined Direct Strength Method (DSM) design expressions for local and distortional buckling [1] as shown in Fig. 2. Note, the FE 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.

As Fig. 2 illustrates, local and distortional cross-sectional slenderness are adequate explanatory variables for parameterizing the peak moment strength of CFS members, and are directly utilized by design specifications [2]. In addition to peak strength, the authors have recently provided new closed-form expressions to predict the pre-peak bending stiffness [7]. The proposed expressions also utilize cross-section slenderness as the explanatory variable and adequately predict the available data as well as provide a significant improvement for bending stiffness prediction when compared to current methods. However, post-peak behavior and ductility, i.e. the full moment-rotation characterization, remains unaddressed.

3. Multi-linear models to represent CFS member moment-rotation behavior

The ductile performance of steel structures benefits greatly from the ability of its members to dissipate energy by means of hysteretic behavior. For a beam the amount of dissipated energy is correlated with the area under the moment-rotation curve. In this section, simplified moment-rotation curves selected by equating the area under the backbone moment-rotation response between available data and simplified multi-linear models are explored. Equivalent Energy Elastic Plastic models [8] with both pre-peak bilinear and trilinear shapes were explored, but rejected due to their inability to include post-peak moment degradation [9]. To include observed response, where in some cases

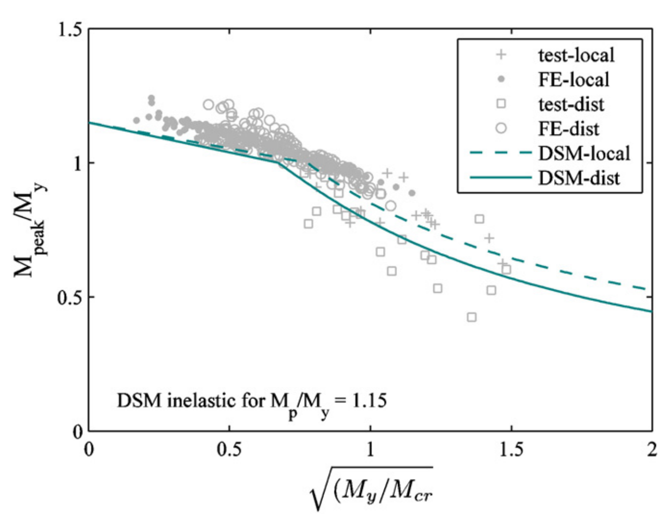

Fig. 2. Correlation of peak moment prediction with DSM (note: every cross-section has a unique M_p/M_y, this influences the inelastic reserve strength prediction, average M_p/M_y for studied sections is approximately is 1.15, strength is only shown for this value.)