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Cyclic axial response and energy dissipation of cold-formed steel framing members

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ABSTRACT

This paper summarizes results from an experimental program that investigated the cyclic axial behavior and energy dissipation of cold-formed steel C—sections structural framing members. Fully characterized cyclic axial load–deformation response of individual members is necessary to facilitate performance-based design of cold-formed steel building systems. Specimen cross-section dimensions and lengths were selected to isolate specific buckling modes (i.e., local, distortional or global buckling). The cyclic loading protocol was adapted from FEMA 461 with target displacements based on elastic buckling properties. Cyclic response showed large post-buckling deformations, pinching, strength and stiffness degradation. Damage accumulated within one half-wave after buckling. The total hysteretic energy dissipated within the damaged half-wave decreased with increasing cross-section slenderness. More energy dissipation comes at the cost of less cumulative axial deformation before tensile rupture.

1. Introduction

Current seismic analysis and design procedures for cold-formed steel (CFS) frame buildings focus on the strength of individual shear wall units [1], e.g., shear walls constructed with CFS steel members sheathed with Structural 1 plywood (4 ply), oriented strand board (OSB), gypsum board, or thin sheet steel or strap bracing. These building subsystems are designed using prescriptive procedures and tabulated values based on shear wall tests.

Research efforts to characterize the response and develop numerical models for CFS lateral load resisting systems typically focus on the response of shear walls to push-over and cyclic tests (e.g., [2–4]). Specific guidance about energy dissipation or strength degradation for these systems and their components (e.g., drag struts, boundary chord studs) is not readily available. The goal of the research summarized in this paper is to experimentally investigate and then quantify the cyclic behavior, and energy dissipation characteristics of CFS axial members. This test data will be implemented in future CFS subsystem seismic numerical models as part of a larger ongoing multi-university research effort [5].

The shift towards performance-based design in earthquake design is creating considerable interest in understanding and controlling building seismic behavior at different seismic hazard levels. To develop seismic performance factors (i.e., R, Ω_0, and C_d) it is necessary to consider suites of ground motions, ground motion intensities and structural configurations [6] with efficient and accurate modeling tools that can simulate structural response of the structural components of CFS buildings. For example, CFS framing could be simulated with hysteretic load–deformation springs in a computationally efficient model as shown in Fig. 1, where the springs are calibrated using experimental data from cyclic tests to represent each of the members and connections. To develop this modeling capability, it is necessary to characterize cyclic behavior and energy dissipation of individual CFS systems, member components and connections.

The experimental program described herein includes twelve cyclic axial tests, 12 monotonic axial tests in compression, and two monotonic axial tests in tension, conducted on common CFS C-section studs without perforations. Cross-section dimensions and specimen length are varied to isolate local, distortional or global buckling, and a loading protocol adapted from FEMA 461 [7] is employed where the target displacements are calculated using the member elastic buckling properties. Local and global buckling slenderness are key parameters influencing energy dissipation of thin-walled steel components, and these trends are explored in a review of existing literature in the next section.

2. Cyclic response of axial members including buckling

Existing cyclic axial tests focused on hot-rolled steel structural sections used for steel-framed buildings and offshore oil platforms. The cyclic axial behavior of globally slender steel members (struts,

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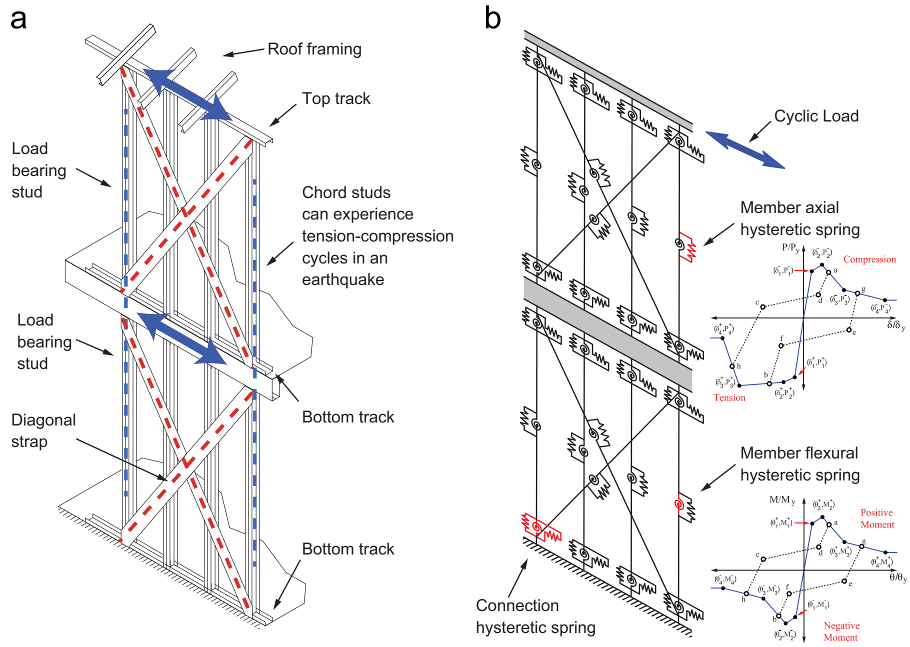

Fig. 1. Cold-formed strap bracing (a) and corresponding phenomenological model (b).

braces) was studied starting in the early 1970s, both analytically and with experimental programs. Hysteretic load–deformation response models for columns experiencing a plastic hinge were developed for finite element models [8–11]. Some of the models included cross-sectional slenderness as a softening parameter [12,13]. The analytical models were combined with experimental data in a few cases to develop semi-empirical equations that predict bracing member fracture life, i.e., the number of cycles to tensile fracture [14].

The viability of these numerical models was established by experiments on structural sections ranging from solid steel bars [8] to hollow thin-walled tubes [15], *W*-sections [16], and angles [17]. A few experiments even considered the influence of cold-bending on energy dissipation [18]. These studies have shown that, inelastic elongation of the members during tensile excursions occurred in a relatively predictable manner [8,16,17]; that tension strength remained fairly constant during inelastic cycles, but compression strength degraded with the number of cycles (implying damage accumulation in compression) [15–17]; when local buckling accompanied global deformation, the member failure mode was typically tensile fracture caused by stress concentrations at a fold [15]; that inelastic deformation of the steel was the key contributor to energy dissipation as compared to inherent material damping [17]; that the total energy dissipation appears to be independent of initial loading direction (tension then compression or compression and then tension) [17]; and, that the amount of total hysteretic energy dissipated decreases as the global slenderness increases [15,17].

Only a few experimental programs focused on energy dissipation from local buckling [19–22]. These studies showed that local buckling compression strength degraded to a constant magnitude with increasing cycles because of inherent post-buckling capacity, which is different and potentially more beneficial to seismic performance than global buckling cyclic behavior where compressive strength decreases to zero as the plastic hinge develops. The experimental program described below explores these post-buckling benefits and documents the cyclic behavior of CFS axial members with a focus on thin-walled member buckling limit states, i.e., global, distortional and local buckling.

3. Experimental program

An experimental program was conducted to study the cyclic response of CFS axial members experiencing local, distortional and global buckling. Cyclic tests were conducted to determine the effects of reversed cyclic loading (i.e., tension and compression) and cumulative axial deformation on damage and hysteretic energy dissipation of members experiencing global, distortional or local buckling. Monotonic tests were performed to establish a load–deformation envelope for comparison to the cyclic test response.

3.1. Specimen selection strategy

Specimens were selected such that their predicted monotonic capacity in compression is governed either by local, distortional or global buckling as predicted by the American Iron and Steel Institute (AISI) Direct Strength Method (AISI-S100-07, [23]). The cross-sections considered, five (5) in total with web widths of 92 mm and 152 mm and nominal thicknesses ranging from 0.88 to 2.58 mm, were chosen from standard sizes as listed in the Structural Stud Manufacturers Association (SSMA) catalog [24]. Cross-section dimensions and length (L) varied to isolate each buckling limit state. Global buckling specimens have a length L=2286 mm, while for distortional buckling tests L=610 mm and for local buckling L=305 mm. The test program included two specimens subjected to quasi-static cyclic displacement and two specimens subjected to monotonic displacement (in compression) per specimen type. Two monotonic tests in tension were included to define the representative tension side load–deformation envelope. Specimen nomenclature is explained in Fig. 2a.

3.2. Specimen dimensions, material properties and elastic buckling loads

Cross-section dimensions were measured at member mid-height using methods described in [25], see Table 1 and Fig. 2b. These values were input to the finite strip eigen-buckling analysis software CUFSM [26] to calculate the elastic buckling loads for