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# Numerical prediction of available rotation capacity of cold-formed steel beams

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ABSTRACT

The rotation capacity of cold-formed steel (CFS) beams has been evaluated through numerical investigation. Studies on different structural levels have been performed. At the beam level, different values of profile thickness have been considered in the range from 1 mm up to 4 mm. In addition, different section shapes have been investigated by increasing the number of flange bends: C-section, broken-section and curved-section, which represents an infinite number of flange bends. At the connection level, a web bolted moment resistant type of connection using through plate has been adopted with different configurations of the through plate dimensions. In web bolted connections without out-of-plane stiffeners, premature web buckling results in early loss of strength. Hence, different configurations of out-of-plane stiffeners have been examined to delay web and flange buckling and to produce relatively high moment strength and rotation capacity. The finite element method results revealed that increasing the number of flange bends will not in all cases enhance the behavior. Meanwhile, the use of out-of-plane stiffeners can increase seismic energy dissipation, strength, initial stiffness, and rotation capacity, when compared with the case without stiffeners.



## 1. Introduction

Cold-formed steel (CFS) sections have several advantages in building constructions, such as the unusual sectional configurations that can be produced economically by cold forming operations, and consequently favorable strength-to-weight ratios can be obtained. Studies have been performed for new profiles such as hat, Z, box and rack sections to predict the appropriate equations that can describe the structural behavior considering the effect of local and distortional buckling [1–3]. The use of CFS sections as main structural elements in building constructions is mainly limited to roof purlins and wall girts, mezzanine floors, storage racks and stud-wall frames with low seismic energy dissipation capacity [4–9]. Generally, moment-resisting frames with CFS sections cannot create plastic hinges in CFS beams; and in turn, cannot produce adequate ductility for high seismic resistance [10–13]. This is attributed to the premature local failures and low strength and stiffness of the CFS beam. The previous studies on behavior of CFS elements and associated beam-column connections in moment-resisting frames under monotonic and cyclic loading are relatively limited [7–9,14–20]. It has been reported that the ductility is mainly provided by the connection (yielding the material around the bolt hole) while the beams and columns remain elastic [7–9]. In recent investigations, conventional back-to-back lipped C-sections possessed a degree of ductile capacity in dissipating seismic energy [14,15]; however, they did not satisfy the required width/thickness limits of design codes [10–13] that aim to delay local buckling after yielding. If CFS beams are used as main dissipative elements in earthquake resistant frame buildings, their ductility needs to be significantly improved by delaying local buckling and allowing development of large plastic deformations. This can be achieved by optimizing the shapes of the CFS sections, using pertinent connection details, and employing stiffeners [21–23]. The use of out-of-plane stiffeners has been examined to improve strength, ductility and energy dissipation [23–26], it has been reported that CFS sections offer good potential as structural elements in the lateral load resisting systems. Furthermore, the capacity and behavior of CFS has been studied and compared with the results of the direct strength method [20,27,28].

This paper presents numerical investigation that underpin the usage of CFS sections as energy dissipative elements for moment-resisting frame buildings. The beam element has been examined and the rotation capacity has been evaluated. The investigation of the beam-to-column connection is considered beyond the scope of study at this stage. A bolted web to through plate connection has been adopted, assuming rigid support for the column and full node coupling at bolt location, as shown in Fig. 1.

## 2. Numerical model

As shown in Fig. 1, the numerical FE model has been established, in ANSYS [29], for 2 m long CFS beams to evaluate the rotation capacity. A web bolted to through-plate connection has been adopted, where the column is substituted with restrained boundary conditions (rigid column). The boundary conditions, loading points and constraints are

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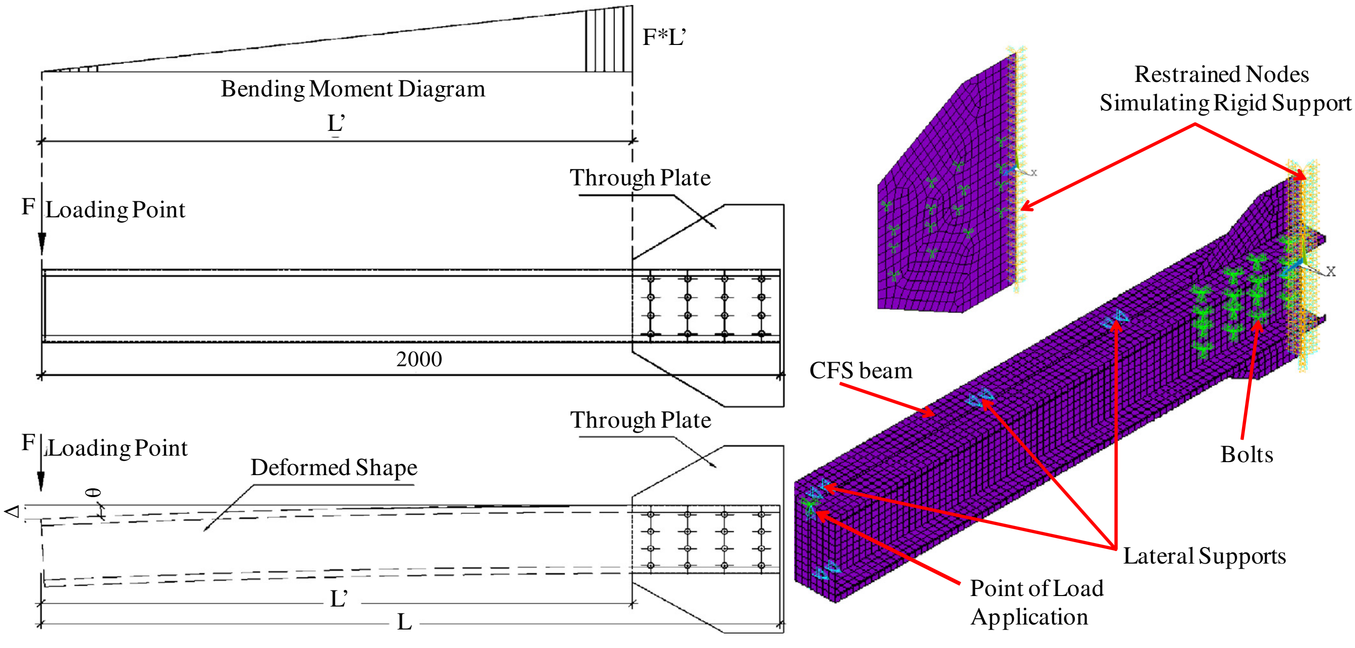


**Fig. 1.** Finite element model and boundary conditions.

shown in Fig. 1. The parameters of the FE model are: element type: 4-node shell element (shell181), mesh size: 20 mm × 20 mm selected upon mesh sensitivity analysis, and material: bi-linear stress-strain behavior (DIN-17,100 ST-37) with Fy = 240 MPa (yield stress), Fu = 360 MPa (tensile stress), E = 210 GPa, (elasticity modulus), Es = E/100 (hardening modulus) and υ = 0.33 (Poisson's ratio). Initial imperfection has been adopted in compliance with the fundamental buckling mode shape attaining a maximum amplitude of [beam height/10,000]. Hence, nonlinear inelastic post-buckling analysis has been performed. The FE model has been established to be as simple as possible, where the bolts that connect the CFS beam with the through plate have been modeled as point ties. Because of this simplification, the tied points, replacing the bolts, do not allow uniform rotation of the bolt group. This simplification is on the conservative side since the critical demand is on the beam and the FE model increases this demand slightly. It is worth noting that the FE model has been designed to enable inelastic behavior in the beam, while the through plate remains elastic. Furthermore, the connector between the beam back-to-back elements has been modeled using rigid beam element (MPC184). The loading point has been selected at the end of the beam, where a pair of vertical stiffeners has been employed to prevent the local failure.

Different values of beam profile thickness have been adopted in the range from 1 mm up to 4 mm. The width/thickness ratios of the compression elements of steel sections are restricted by design codes to certain limits to avoid local buckling before yielding [10–13]. These width/thickness ratio limits are not possible to be satisfied by typical CFS sections. Local buckling of CFS sections, however, can be delayed by changing the geometry of the section, rather than restricting the width/thickness ratio. Hence, in this study three section shapes (geometries) have been considered by increasing the number of flange bends, namely: channel section (C-section), broken section and curved section, which represents an infinite number of flange bends. It is worth noting that both broken and curved sections impose more fabrication effort, as experienced during fabrication of the test specimens. Hence, the C-section is considered to be more practical compared to broken and curved sections. Fig. 2 illustrates the adopted CFS beam section

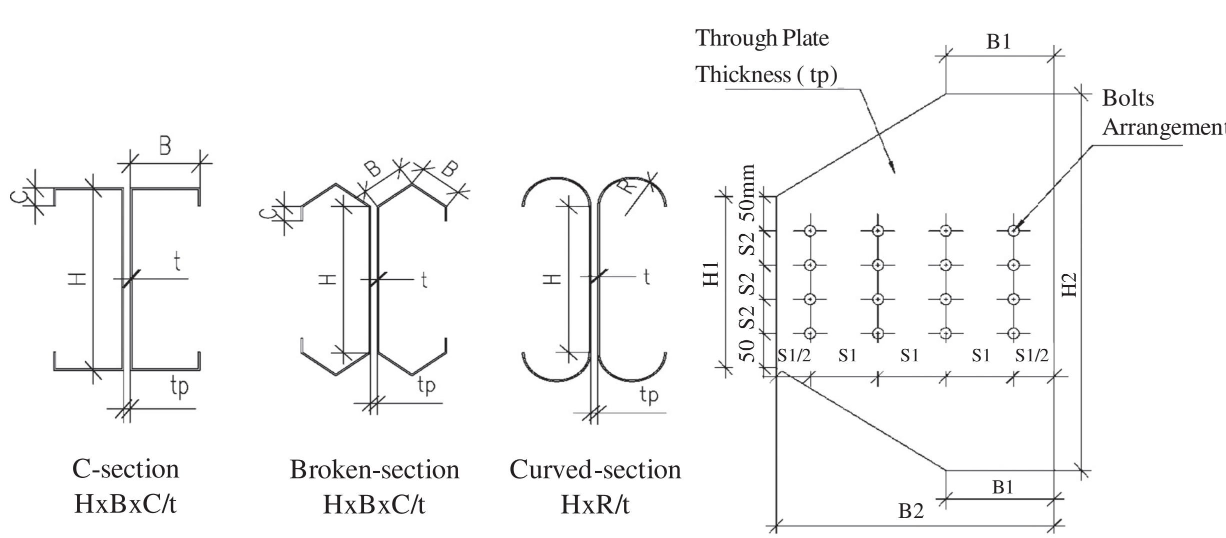


**Fig. 2.** CFS beam section shapes and through plate dimensions.