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Experimental work on cold-formed steel elements for earthquake resilient moment frame buildings

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ABSTRACT

This paper presents an experimental investigation on the use of thin-walled cold-formed steel (CFS) sections as energy dissipative elements for earthquake resistant moment frame multi-storey buildings. The tests were performed on six bolted beam-to-column connections, using through plates and curved flange beams with different types of out-of-plane stiffeners in the connection region. The hysteretic behaviour of the CFS connections shows high seismic energy dissipation capacity and sufficient ductility to satisfy code requirements for seismic design. The use of out-of-plane stiffeners inside the beams in the connection region results in improvement of the moment–rotation behaviour of the connection by up to 35% in strength and 75% in ductility. Mobilising connection slip after the elastic cycles provides highly stable hysteretic behaviour and an increase of up to 240% in energy dissipation capacity. The tested connections can be classified as rigid with partial or full strength depending on the connection stiffeners.

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1. Introduction

Experimental work on monotonic and cyclic behaviour of components and elements of CFS moment-resisting frames (MRFs) is very limited [1–4]. Premature local failures are prevalent in common CFS sections because of their thin-walled elements. One solution to avoid premature local failures and to provide ductility in bolted CFS connections is to mobilise slip and bearing action of bolts while beams and columns remain elastic [1]. This limited source of ductility restricts such structures to one-storey dwellings [1]. For seismic design of multi-storey buildings, there is a need to dissipate large amount of energy through plasticity in the beams rather than just yielding the material around the bolt holes.

There are research studies [2,3] showing that by using appropriate connection details for CFS beam-column connections, such as gusset plates, relatively high moment resistance can be developed in CFS double back-to-back channel sections. In this type of beam-to-column connection however, no ductile capacity was achieved after reaching the peak bending moment. The general assumption is that CFS beams with thin-walled elements cannot develop plastic hinges, thus cannot be used for high seismicity areas [5–7].

In a recent study by the authors [4] conventional double back-to-back channel beam sections integrated within topping concrete were shown to possess a degree of ductile capacity in dissipating seismic energy by achieving rotations larger than 0.04 rad; satisfying the requirements for special moment frames [7]. However, they did not satisfy the required width/thickness limits of design codes [5–7] which aim to delay the local buckling after yielding. This research also showed encouraging results for through plate type of CFS beam-to-column connections [4].

If CFS beams are designed as the main energy dissipation components in seismic resistant MRFs, the ductility capacity of the beams with thin-walled elements must be improved. The first step is to delay local buckling as much as possible to enable plastic deformations. Curved flange beam sections were developed by the authors [8–10] by introducing more bends in the flanges (Fig. 1a), a step-by-step process that ultimately led to significant increase in moment resistance, stiffness and ductility, compared with flat flange beams.

In this study a web bolted moment resistant type of connection is used for CFS beam-column connections. This type of connection has already been examined as lapped connections in portal frames [11,12]. The main components of the beam-column MR connection are welded cross through-plates which can be bolted to separate beam and column sections, as shown in Fig. 1b. Previous research [8–10] has shown that web bolted through-plate beam-to-column connection produced a lower level of ductility and strength than that of a theoretical fixed-end beam. Web buckling adjacent to the first line of bolts at the beam-through plate connection was identified as the main reason for premature loss of strength [8–10].

An optimum combination of vertical and horizontal out-of-plane stiffeners has been identified for the web bolted CFS connec-

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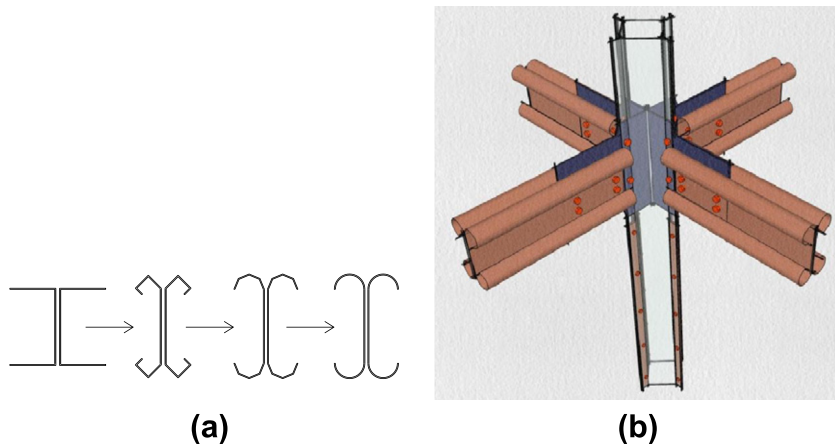

Fig. 1. (a) Step-by-step development of curved flange sections and (b) CFS beam-column connections: diamond column, cross through-plates and curved flange beam [8–10].

tions to increase both the strength and ductility [8–10]. Minimum number of vertical stiffeners has also been identified for easier implementation and for connections where ductility demand can be met by a reduced number of stiffeners rather than a full (optimum) set of stiffeners [9,10].

The experimental study presented in this paper investigates the concept of CFS thin-walled curved flange sections as seismic energy dissipative elements for moment frame buildings using full and minimum sets of out-of-plane stiffeners. It also aims to examine if curved flange CFS sections can produce full plastic moment (M_p) sustained at large rotations similar to Class 1 cross sections in Eurocode 3 [6] and larger than 0.04 rad required for special moment frames in AISC Seismic Provisions [7].

2. Testing arrangement and specimen details

Two specimen types with different thickness (A) 3 mm, and (B) 4 mm, with three different out-of-plane stiffener configurations (A1, A2, A3, and B1, B2, B3) were used in the experimental investigation (Table 1). The nominal dimensions of the components of the test specimens are shown in Fig. 2.

According to the FE analysis presented previously by the authors [8–10], the connections with full stiffeners (used for Specimens A3 and B3) produce a significant increase in both strength (∼40%) and ductility (∼100%) in comparison with the connections without stiffeners (Specimens A1 and B1) which were used for bench marking and comparison purposes. The difference between the boundary conditions of FE models and the test specimens is that hot-rolled back-to-back channels were used in the tests instead of CFS columns. This was easier to manufacture and install into the testing rig. This change was supported by the results of the FE analyses, which showed no yielding or large deformation in the CFS columns [9,10].

For all specimens, the connections were designed using the requirements for slip-critical joints given in AISC Specification for Structural Joints [13]. The design slip resistances of the farthest bolts (R_n) in the connections were calculated by assuming slip coefficient of $\mu = 0.5$ for uncoated blast-cleaned steel [13] and applying a pretension force $T_m = 67$ kN for the beam-to-through plate (B–T) and $T_m = 53$ kN for the through plate-to-column (T–C) connections, approximately equal to 42% of the tensile strength of the bolts (60% of the 70% tensile strength, given in the Specification [13]). This resulted in design resistance (R_n), higher than the required resistances (R_{req}) for specimens type A, but just below the required resistances (R_{req}) for specimens type B. Therefore, it was expected that if the beam plastic moment was mobilised in the tests of specimens type B, slipping at the connections would be triggered.

Table 1
The specimens configurations.

Specimens	Beam thickness (mm)	Connection stiffeners	Connection type
A1	3	No stiffeners	Slip-critical
A2	3	Partial (minimum) stiffeners	Slip-critical
A3	3	Full (optimum) stiffeners	Slip-critical
B1	4	No stiffeners	Slip-critical
B2	4	Partial (minimum) stiffeners	Slip-critical
B3	4	Full (optimum) stiffeners	Slip-critical

2.1. Test set-up

Fig. 3 shows a drawing and a photo of the test set-up. A brief description of the design specifications for the test set-up components are given in Appendix A.

2.2. Instrumentation

Strain gauges (SGs), inclinometers and LVDTs were placed at different locations of the through plates, beams and columns as

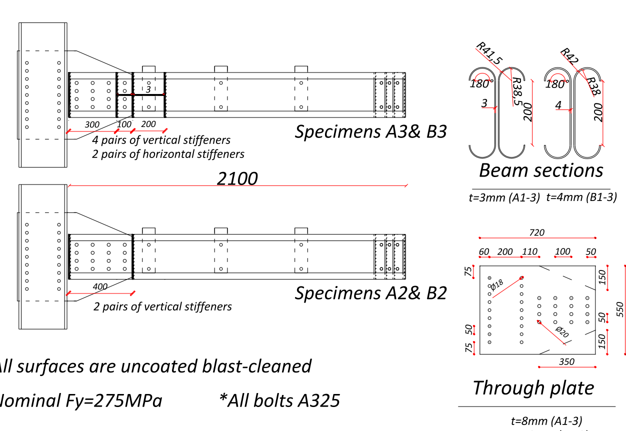

Fig. 2. Dimensions and configuration of the test specimens.

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