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Analytical and experimental study on the seismic performance of cold-formed steel frames

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ABSTRACT

This study aims to investigate the seismic performance of an innovative cold-formed steel (CFS) moment-resisting frame experimentally and analytically. A half-scale CFS moment-resisting portal frame was tested under static monotonic loading until failure. The frame consisted of two box-shaped columns (face-to-face channels connected with inside plates), a back-to-back lipped channel beam section and fully moment-resisting CFS bolted connections. During experimental tests, damage mostly concentrated at the top and bottom of the CFS columns due to the web crippling of the channels close to the connections, while no fracture or obvious slippage was observed at the connection zones. A detailed Finite Element (FE) model was developed using ABAQUS by taking into account the material non-linearity and geometrical imperfections. The lateral load-displacement behaviour, ultimate strength and failure modes predicted by the model were in very good agreement with the experimental results. The validated FE model was then used to assess the effects of key design parameters on the lateral load capacity, ultimate displacement, energy dissipation, ductility, and ductility reduction factor of the frame. It is shown that the proposed system can provide good seismic performance subjected to the appropriate design of the main structural elements. Increasing the axial load ratio of the columns by 50% resulted in 26%, 62%, and 50% decrease in the ultimate lateral load, energy dissipation capacity, and ductility ratio of the CFS frame, respectively. However, the energy dissipation capacity and the ductility ratio of the proposed system increased significantly by decreasing the width-to-thickness ratio of the columns.

1. Introduction

Cold-formed steel (CFS) structural elements have seen a dramatic expansion in the range of applications over the past decade. CFS systems are currently used in structural, mechanical and aeronautical engineering. Compared to their hot-rolled counterparts, CFS members are often more economical and efficient due to their light weight, ease and speed of construction and a greater flexibility in manufacturing. However, the inherently low buckling resistance of thin sections results in relatively low strength and ductility in CFS structural elements, which may limit their performance under extreme loading events such as strong earthquakes. While the seismic performance of CFS structures has been previously investigated [1–5], majority of these studies have focused on the behaviour of shear wall panels as the main lateral load-bearing system. However, using moment-resisting CFS frames can reduce the reliance on the fixed shear wall panels and therefore provide higher flexibility for space planning (e.g. opening sizes and proportions) and future alterations. On the other hand, compared to shear wall systems, using moment-resisting CFS frames can potentially improve the energy dissipation capacity and the seismic performance of the CFS structures under strong earthquakes.

The structural performance of moment-resisting connections (named eaves and apex joints) with different bolt group sizes were investigated by Lim et al. [6]. It was shown experimentally and numerically that the strength of web bolted moment resisting connections between cold-formed steel channels can be controlled by the length of the bolt groups. In addition, the Direct Strength Method (DSM) adopted in North American design specifications [7], was further developed to predict the moment capacity of the connections with short and long bolt group length [8]. Sato and Uang [9] showed that the ductility capacity of CFS special bolted-moment frames defined by AISI S110 standard [10], can be improved through bolt slippage and bearing in bolted beam-to-column moment connections. However, it should be noted that AISI S110 [10] standard has now been supressed by AISI S100 [7].

Several studies have also investigated the seismic performance of bolted-moment connections using gusset plates [11–14], where the experimental results are used to validate different modelling techniques (e.g. detailed FE modelling, equivalent spring elements and internal force distributions) for prediction of stiffness and strength of the connections. It should be noted that based on the current AISI S400 standard [15], CFS bolted moment frame systems are expected to withstand

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inelastic friction and bearing deformation at the bolted-beam to column connections. The required strength of beams and columns in the CFS moment resisting frame system shall be determined from the expected moment developed at the bolted connection, using the column shear corresponding to the slip strength of the bolt group and the connection bearing component of column shear corresponding to the design storey drift.

Lim et al. [5] proposed a simplified beam element to model the structural behaviour of CFS portal frames and compared the FE results with those of full scale laboratory tests. It was shown that the deflection due to bolt-hole elongation has the most significant contribution to the total beam deflection. In another study, Sabbagh et al. [16] performed an experimental investigation to study the performance of two full scale one-storey frames made of cold-formed steel members under gravity and lateral cyclic loads. The results of their study in general indicate that CFS moment-resisting frames exhibit good seismic performance subjected to the appropriate details for the main structural elements.

In general, CFS moment-resisting frames investigated in the aforementioned studies provided a better ductility and energy dissipation capacity compared to conventional CFS shear wall panels. However, more studies are required to investigate the non-linear seismic performance of CFS moment-resisting frames using different types of connections and design parameters. In this paper, a CFS moment-resisting frame with box-shaped columns and back-to-back beam channels is laterally pushed monotonically until failure. Two different types of connections are used in this research (i): column-to-base connection fabricated with hot-rolled angle cleats bolted to the CFS column and base, (ii): beam-to-column connection assembled by gusset plates connected to both beam and column elements by using bolts. The failure mode of the tested moment-resisting CFS frame is identified and the structural behaviour of the frame is described step-by-step using a lateral load-displacement curve. A detailed FE model is developed by considering material nonlinearity, geometrical imperfections and accurate behaviour of the bolts. The results are then validated against the experimental response of the frame. Finally, a comprehensive parametric study is carried out to investigate the lateral load capacity, ultimate displacement, energy dissipation capacity, ductility, and ductility reduction factor of the CFS frames with different axial load levels and width-to-thickness ratios of the columns. The results are used to assess the adequacy of the proposed system is seismic regions.

2. Experimental study

2.1. Specimen description and preparation

The aim of the experimental program in this study was to provide information on the seismic performance of an innovative CFS moment-resisting frame system by focusing on the lateral load-displacement behaviour and the failure modes. The CFS moment-resisting frame was a half-scale representative of a moment-resisting frame with the height and the span length of 3000 mm (see Fig. 1(a)). The cross-sectional dimensions and plate thicknesses were scaled so that the lateral stiffness of the scaled frame remains similar to the original frame. The CFS sections were formed by a press brake machine using galvanized steel sheets. However, it should be noted that generally cold-formed structural sections are manufactured by rolling process, causing residual stress and strain hardening of the material especially in the corner zones. The columns were built-up box-shaped sections consisting of face-to-face plain channel sections and two inside plates as shown in Fig. 1 (b) and (d).

The general design philosophy adopted was to provide rigid moment resisting connections using a strong column weak beam approach. The box-shaped columns provide a practical design solution for moment-resisting joints in 3D frames. In this study, the inside plates were connected to the channels' flanges by means of #8 bolts as shown in Fig. 1 (a). Using inside plates offered the following advantages: (i) helped the CFS column element act as a unified box section, and (ii) eliminated the need for welding to shape the box-shaped columns, which is especially important considering the low weld-ability of galvanized steel sheets in comparison with normal ST37 steel sheets [17]. It should be noted that welding of CFS sections with low thickness could also cause perforation and damage in the channels.

The steel grade of S275 was used for CFS framing members with the specified minimum yield strength and tensile strength of 275 and 370 MPa, respectively. The beam section consisted of two back-to-back CFS lipped channel sections with the dimensions of $100 \times 50 \times 2$ mm and $L = 1330$ mm length. The CFS lipped channels were connected by two rows of #8 bolts as shown in Fig. 1 (c). Two back-to-back angle-shaped sections were used as a gusset plate to transfer vertical forces and bending moments [6,12]. The gusset plates ($250 \times 50 \times 4$ mm) were connected to the beam and column elements by using fourteen #8 and #10 bolts (grade 8.8), respectively. Figs. 1 (a) and (b) show the configuration of the beam to column connections in more details. It should be noted that, due to inaccessibility, conventional nuts could not be used inside the box-shaped columns. Thus, the inside plates were threaded at the location of the holes to fix the connection bolts. It should be noted that this type of bolts can provide higher strength compared to the conventional self-tapping screws, which is especially important for moment resisting frame connections in seismic regions. Standard washers were located under the head of all connection bolts and nuts to prevent rotation of the bolts and deformation of the thin material in the vicinity of the bolt holes. The diameter of all bolt holes was fabricated 1 mm larger than the nominal diameter of the bolts.

The column base rigid connections were fabricated using two pairs of hot-rolled angles with dimensions of $80 \times 80 \times 8$ mm. The hot-rolled angle sections were bolted to an I-shape base beam by using M16 bolts, grade 8.8. The base beam was then welded to the rigid floor of the laboratory (see Fig. 1 (d)). It was previously shown that using hot-rolled angle cleats for column-to-base bolted connections, in general, can provide high stiffness and rigidity [18]. To prevent premature failure modes and enhance the capacity of the column-to-base connection, the flanges of the angles were joined by full penetration welding with 10 mm thickness as shown in Fig. 1 (d).

2.2. Experimental test setup

The experimental test setup used in this study is shown schematically in Fig. 2. A 600 kN hydraulic actuator operating in displacement control was utilised to apply horizontal displacement at the top of the column. A thick steel plate was attached to the end of the hydraulic actuator to prevent premature local damage at the place of loading. Furthermore, to ensure that CFS moment-resisting frame has no out-of-plane deformation and to prevent unexpected instabilities and torsional buckling of the test specimen, a lateral bracing was placed at the end of the beam element as shown in Fig. 3. The contact surfaces between the CFS beam and the lateral bracing system were lubricated to avoid friction.

The test was conducted under displacement control conditions, where the monotonic lateral load was gradually applied to the top of the frame columns until failure occurred. To obtain a quasi-static loading procedure, the load was applied at a gradual rate of 0.1 mm/s to let the structure deform in a ductile manner and also to help identifying local damage in the structural elements and connections.

It should be noticed that that the CFS moment resisting frame would experience some stiffness and strength degradation under cyclic loading, which cannot be captured by the static monotonic tests conducted in this study. However, cyclic loading tests on CFS moment resisting connections indicated that this degradation is not significant if the plate thickness is >3 mm, even when the local elastic buckling occurs before yielding of the cross-section [11].