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Experimental And Numerical Investigation Of Lateral Performance Of Moment-Resisting Frames Made Of Cold-Formed Steel Sections

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***Abstract*:** *This paper concentrates on the structural strength and behaviour of cold-formed steel (CFS) moment-resisting frame with boxed columns to the back-to-back lipped channel beam. A monotonic test was conducted at CFS moment-resisting frame. The experimental set-up, column and beam components, connection configurations and material properties are first introduced, followed by a detailed account of the results and observations from the test. It is noted that the failure mostly occurred at the top and bottom of columns due to the local buckling. Finite element modelling (FEM) using ABAQUS was developed to investigate the behaviour of CFS moment-resisting frames. The results obtained from the test specimen were verified against those FEM results.*

Keywords: *Cold-formed steel; Moment-resisting frames; Monotonic loading; Finite element analysis (FEA) model; Ductility; Energy absorption*

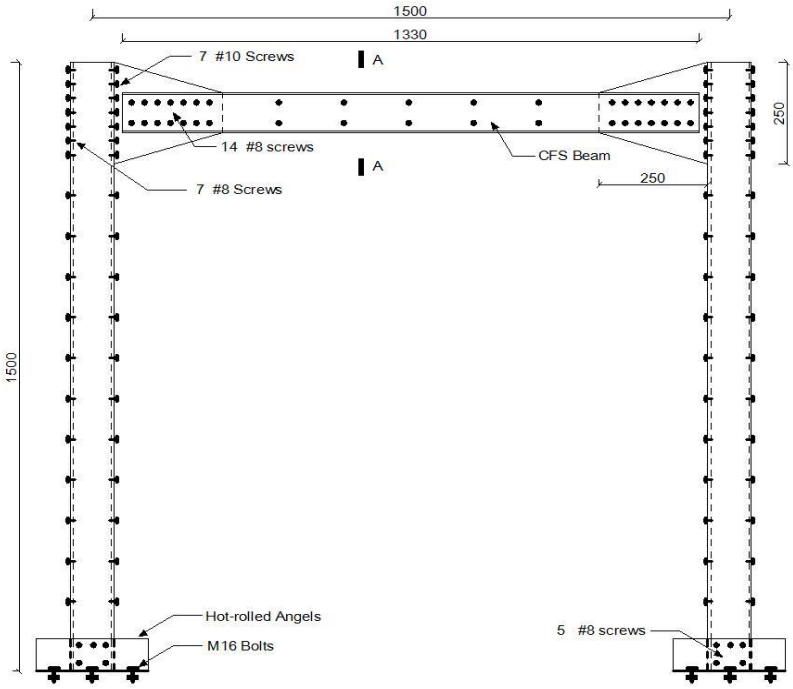
1. Introduction

Structural sections processed from thin sheet steel by cold rolling, brake pressing or folding, which called cold-formed steel sections (CFS sections) are extremely widespread in use at the present time. Today, usage of CFS frames has developed in the residential low level or high level. The advantage of CFS, such as lightweight, durable, quickly made and assemble, safe connections and 100% recyclable, are probably important reasons for this growth in use. CFS frames in the seismic regions should take advantage form the appropriate resistance system against lateral load, and also they must insure reliable connections and ductility. Two Kinds of CFS frames are: (i) braced frames, and (ii) Moment-resisting frames. Braced frames are more conventional than moment-resisting frames because of the scarcity of researches for a moment-resisting frames. The use of cold-formed steel sections (CFS sections) as main structural components in buildings is mainly limited to braced frames with low seismic energy dissipation capacity. Hence, For CFS structures, there is a need to develop CFS moment-resisting frames in the same way as for hot-rolled steel moment-resisting frames. In addition, braced frames can make some architectural problems and also lack of the frames and voids with big span. Because in such frames, the distance of studs is considered very near to each other (about 1m) in order to prevent global buckling of thin bracing member due to its weight as well as the existence of bracing members. These arguments lead to increase research about moment-resisting frame and use it excessively. Investigation on seismic performance of different types of moment resisting frame identified the demand for serious improvement in the connections and members to achieve a more ductile behaviour [1, 2]. The lateral design of CFS moment-resisting connections can be found in the AISI S110 standard [3]. Several state of the art reports or papers were published recently on CFS structures, such as Dundu [4], Wang et al. [5], and Chuquitaype et al. [6]. Other investigations [7, 8] have shown that by using appropriate connection details, such as gusset or through plates, and also optimum stiffener in beam-to column connections, relatively high moment resistance can be developed in CFS beams.

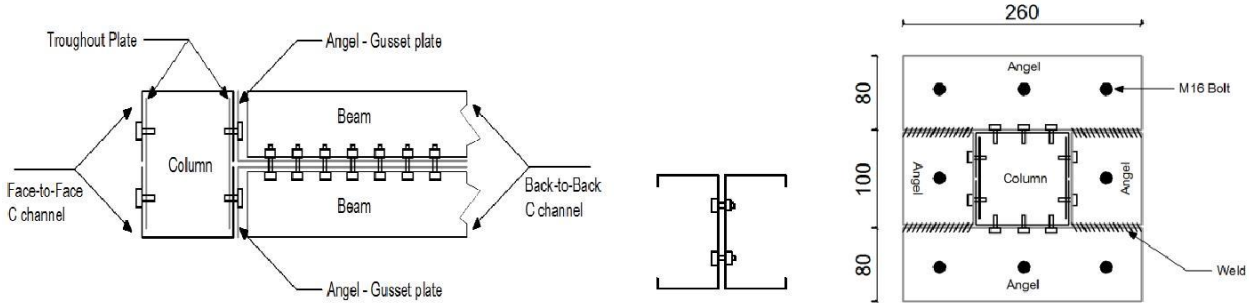
2. Experimental study

2.1. Specimen description and preparation

A 2-D flexural frame made of cold-formed steel (CFS) sections have been tested in the structural laboratory of Amirkabir University. The aim of this test is to investigate the performance of box columns of this frame under lateral load by focusing on failure modes. The height of the column and span length were respectively 1500×1500 mm. The front view of the frame is shown in Fig. 1.



(a) Frame elevation.



(b) Fabrication of beam to column connection (c) A-A. Beam section. (d) column-to-base connection.

Fig. 1: Moment-resisting frame configuration

All Box-shaped columns included two components:

- Face-to-face channel sections
- Throughout plate

To connect face-to-face channel sections and also to insure unified performance of it, throughout plate was used. Throughout plate were connected to channel sections flanges by two #8 screws in each row with different vertical distance shown in Fig. 1 (a). Throughout plate has brought several advantages for CFS moment-resisting frame:

- To Increase the thickness of the columns in a direction which frame has been pushed, so that the maximum flexural moment would occur there.

- To avoid using weld, because of the lower weldability of galvanized steel sheets in comparison with normal ST37 steel sheets [9].
- Box shaped column was suggested to enhance versatility and practicality of this kind of moment-resisting frame and to provide the possibility of designing the frame in 3D.

The beam is made from double back-to-back coupled cold formed lipped channel sections were connected together by #8 screws as shown in Fig. 1 (c). Back-to-back angle-shaped sections were used as a gusset plate which was connected to beam by #8 screws and to columns by #10 screws Fig. 1(a). Also, Fig. 1(b) shows the configuration of beam to column connection in detail. It should be noted that nuts could not be used inside of the boxed-shaped column because of inaccessibility. Thus, before fabrication of the column, throughout plate was a thread where holes were located to ensure that throughout plate perform as nuts and make the connection fully fixed. Rigid connection of column base was established by two pairs of hot-rolled angles. To ensure the unified function of angles, enhance the capacity of column to base connection and prevent premature failure at the connection of column to base, flanges of angels are welded together by penetration weld with a thickness of 10mm which is shown in Fig. 1(d). Summary of the test is given in the that includes geometric details

TABLE I

of connection and also the size of beam and column components.

TABLE I: Summery of material properties

Member	Dimension (mm)	Length (mm)
Face-to-face channel (column)	100×50×3	1500
Throughout plate	100×4	1500
Back-To-back lipped channel (Beam)	100×50×2	1330
Gusset plate (Angel-shaped)	250×100×4	250
Angle base plate	80×80×8	100 , 260

2.2. Material properties

Prior to testing CFS moment-resisting frame, material properties must be determined. Three coupons testing was prepared and tensile test was performed on them by extensometer. As a result, the average material yield strength and ultimate strength of materials for CFS components used (Gusset plates, throughout plates, Channels) were determined, as are shown in TABLE II.

TABLE II: Material properties

	#1	#2	#3
0.2% proof stress (MPa)	265.36	269.45	258.9
F_{max} (UTS) (MPa)	313.54	322.88	314.18
Strain at fracture	24.1%	26.3%	24.8
Thickness (mm)	1	1	1
E-Modulus (GPa)	197.6	199.1	194.2

3. Testing Procedure And Results

A 600 KN Hydraulic actuator operation in displacement control was used to apply horizontal displacement at the top of the column. When the lateral load was gradually applied at the top of the left column, as it was expected it was found that CFS moment-resisting frame failed at the components of the column. Obvious buckling deformation formed at the top and bottom of columns, as shown in the Fig. 2, and the simulated failure modes were given in this figure too. There were no obvious failures base connection due to opening up angles on the tension side and closing of the angles on the compression side. The base was significantly stiffened due to using strong angles at the column-to-base connection, hence, it caused source of failure to shift from the angles to the column. After testing, it was noticeable that not only any of the column-to-base connection angles did not experience bolt bearing deformation around the bolt holes, but also there were no shear failure and pull-out in screws connecting frame members. It should be noted that no lateral torsional buckling of the beam was

observed during the monotonic loading test.

Fig. 2: Buckling deformation at top and bottom of the column

Lateral load (P) versus lateral displacement ( ) curve of the CFS moment resisting frame specimen is shown in Fig. 3, in which some points have been highlighted. For each phase a brief description with significant photographs of the occurred phenomenon is given. The first phase (0-A) corresponds to the elastic performance of the frame. The second phase (A-B), which corresponds to the material nonlinearity stage, the frame was pushed up to 55mm, and the applied load on the frame reached to 34.5KN, so denoting the attainment of first yielding in the section before the occurrence of local instability. In the third phase (B-C) for a displacement equal to 55mm, local buckling at the top of the right column at the location of column-to-gusset plate connection occurred. This local buckling creates a softening branch in the diagram, so it leads to strength deterioration and stiffness degradation for CFS frame. The column showed a very strong deformation in the buckled zone, thus, because of that there appears a small increase in the lateral load capacity of the CFS frame. In the fourth phase (C-D), the displacement at the top of the frame reaches to the 300mm. Once this displacement was exceeded, the specimen experienced an unstable decreasing behaviour (phase C-D), and also Increasing the applied displacement, local buckling progressively increased and involved the channel's web forming a box-shaped column close to the column-to-base connection.

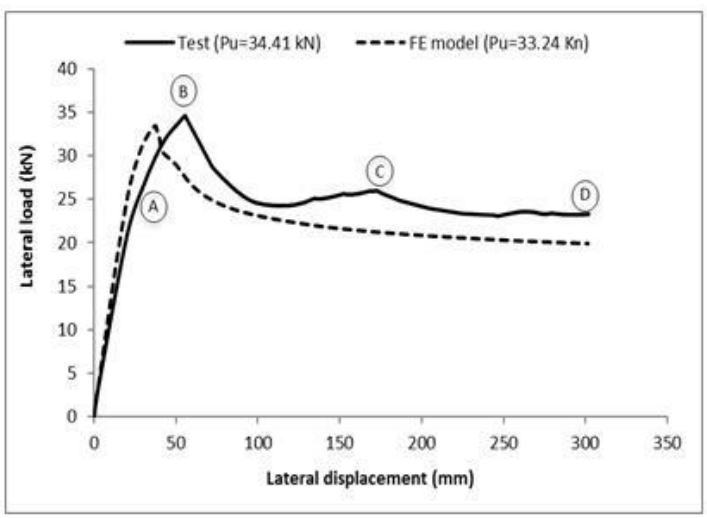


Fig. 3: Lateral load versus lateral displacement curves for test and FE model.

4. Finite element analysis

The main objective of the proposed 3-D finite element models is to simulate the CFS moment- resisting frame behaviour and also validate experimental test. The FEA package ABAQUS [10] were used in all nonlinear monotonic (pushover) as well as axial static load. Shell element would be more efficient in modelling frame members, and element type was 4-node standard linear with reduced integration points. It is found that mesh size of 15×15 mm for shell elements can obtain accurate results, shown in. In order to accurately simulate exact performance of CFS moment-resisting frame, the main five components of the frame need to be modelled. These components are the column channels, beam channels, throughout plate, angle-shaped gusset plate, and base plate angles as well as screws. Elastic-plastic material behaviour provided by ABAQUS (using the plastic option) allows a bi-linear or multi-linear stress-strain curve to be used. The model parameters for bi-linear stress-strain curve are: yield strength (F_y = 265 MPa), modulus of elasticity (E = 200 GPa), poisson's ratio (ν = 0.3), ultimate tensile stress (F_u = 313 MPa), second modulus (Es = 0.1 E). Both material and geometric nonlinearities are included in this study. The screw is modelled by using beam connector which constrains all 3 degrees-of freedom of one node to an adjacent node. In a previous study, Sabbagh [11] considered FE modelling of screws including slip. However, it was not observed slip of screws during the experimental test; thus, slip in the connections could be ignored. All nodes of the base angles at the location of bolt stub were fully constrained by Tie command in ABAQUS; also out-of-plane displacement was prevented by lateral restrains.

Fig. 4: Finite element model of CFS moment-resisting frame

4.1. Verification of FEA model

It should be noted that only the experimental test results presented in this paper were used to verify the FEM model. Nonlinear inelastic analysis (pushover) was performed by using the standard RIKS arc-length method in ABAQUS, which consider stiffness degradation due to buckling. As shown in Fig. 3, The FE curves matched well with the lateral monotonic load (P) versus lateral displacement (Δ) relationships with experimental results. Numerical and experimental P- curves have a good agreement on elastic performance, but there is a bit different in plastic behaviour. Because the phase difference between local buckling at the top and bottom of the column can be seen in the experimental curve, but local buckling at the top and bottom of the column occur simultaneously in FEA. The ultimate lateral loads of CFS moment-resisting frame obtained from the test and finite element analysis by ABAQUS, as well as the load–displacement curves were shown in Fig. 3. It should be noted that, there were seldom experimental results at CFS moment-resisting frames had been published in the past.

5. Conclusion

Based on the experimental test and FEA of CFS moment-resisting frame, the following conclusions can be drawn:

- The experiment shows that CFS moment-resisting frame has good earthquake resistance. However, there was some failure observed in the CFS moment-resisting frame test. The local buckling at the top of the columns is the major reasons to induce frame failure. After that, another failure phenomena, mainly occurred bottom of the column.
- According to observations of CFS moment-resisting frame test, there were not any bearing failure of CFS members around the screws and bolts. Also, there appear no pull-out and shear failure of screws.
- The FE modelling which was used in this paper to validate experimental test was able to reasonably predict the lateral load versus lateral displacement curve of the CFS moment-resisting frame, and the ultimate lateral load.
- Considering the aforementioned results and comparing those with the results of hot-rolled steel moment-resisting frames, it is concluded that the performance of CFS moment resisting frame under cyclic loads is not satisfactory. That is because, even after not occurring some undesirable failure modes like screw pull-out and shear failure mode in the screws, there still remains the buckling failure mode in the box-shaped column elements, which is an undesirable failure. To solve this issue, it can be suggested to use a stiffener inside of the column at location of column-to base and gusset plate connections. The analytical results by using the FEM model with stiffener will be presented in another accompanying paper.

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