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McCrum, Daniel P.; Simon, Jordan; Grimes, Michael; Broderick, Brian M.; Wrzesien, Andrzej; Lim, James

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Manuscript title: Hybrid seismic testing of cold-formed steel moment resisting frames

Authors: Daniel P. McCrum¹, Jordan Simon², Michael Grimes³, Brian M. Broderick³, Andrzej Wrzesien⁴ and James Lim⁵

Affiliations: ¹School of Civil Engineering, Newstead Building, University College Dublin, Dublin 4, Ireland; ²Ecole D'ingénieurs Centre des Etudes Supérieures Industrielles, 93 boulevard de la Seine, BP 602, 92006 Nanterre Cedex, Paris, France; ³Department of Civil, Structural & Environmental Engineering, Trinity College Dublin, Ireland; ⁴School of Engineering and Computing, University of the West of Scotland, Paisley, High Street, PA1 2BE, Glasgow, UK and ⁵Building 401, Level 10, Room 1012, 20 Symonds Street, Auckland 1010, New Zealand

Corresponding author: Daniel P. McCrum, School of Civil Engineering, Newstead Building, University College Dublin, Dublin 4, Ireland. Tel.: +353 (0) 1 716 3214.

E-mail: daniel.mccrum@ucd.ie

Abstract

The true seismic performance of three full-scale 2.2m high by 3.2m long cold-formed steel (CFS) moment resisting frame structures is investigated for the first time in this paper. In general, shear wall type CFS structures have performed well during earthquake events in the past, however portal frame CFS type structures are little investigated. Limited cyclic testing of CFS moment resisting connections/frames has shown the connections/frames perform well. However, these cyclic tests do not take account of inertia forces of the framing structural system. The seismic performance of CFS portal frames was investigated for the first time under true seismic loading using the hybrid test method. Results show that the frames perform well and under extreme loading fail through local buckling of the column section at the column to haunch connection. This failure mechanism needs to be prevented as it does not align with the strong-column weak-beam design philosophy.

Keywords: Steel structures; Structural frameworks; Seismic engineering; Fatigue

1. Introduction

Low-rise (1-2 storeys) cold-formed steel (CFS) buildings can perform well in seismic events. One possible reason for the good performance of CFS structures is that they tend to be low-rise domestic buildings and relatively light in mass. The inertia forces experienced during an earthquake can be low in comparison to standard construction types. This statement is somewhat of a simplified generalisation as the response is also dependent on many other factors such as the frequency content of an individual earthquake and the natural frequencies of the structure. The main reason low-rise CFS structures perform well is that they have redundancy and overstrength (Rondal & Dubina (2005)). Typical low-rise CFS construction consists of studded shear walls with multiple vertical and horizontal structural elements sheathed together with plasterboard. Most of the research to date on CFS structures has focused on studded shear wall type of construction. This paper aims to investigate the true seismic performance of CFS moment resisting frames for two main reasons; (1) these structures have the potential to be used in multi-storey building structures, and (2) the seismic behaviour of CFS moment resisting frames have not been well investigated.

Most of the cyclic testing of CFS moment resisting frame structures has not focussed on the full frame, however focused solely on the moment resisting connections (Uang et al. (2010), Bagheri Sabbagh (2012 & 2013)). Cyclic testing is an easily repeatable test method that simulates the low cycle fatigue experienced by a structure or component during a seismic event. More recently, researchers have investigated the cyclic performance of entire CFS moment resisting frames (Li et al., (2014), Mojtabaei et al., (2018) & McCrum et al. (2018)). Inertia forces are not excited during a cyclic test as it is performed at a slow rate of loading. Therefore, a true dynamic test is required to understand the full dynamic response of a structure. Several shake table tests have been performed on CFS shear wall/braced frame structures (Kim et al., (2006), Shamim et al., (2013), Peterman et al., (2016a&b) & Fiorino et al., (2017)) but none have been performed on CFS moment resisting frame structures. However, still no full-scale seismic test of a CFS moment resisting frame has taken place. The seismic tests performed in this study were part of a larger research project to investigate the seismic performance of CFS moment resisting frames through monotonic and cyclic testing (McCrum et al., (2018)). The cyclic tests by McCrum et al., (2018) of the same structure investigated in this paper demonstrated that the CFS moment resisting frame performed well under low cycle fatigue loading by exhibiting stable hysteretic response, good hysteretic energy dissipation capacity and ductility. The failure of the column sections through local buckling and tearing was observed. However, no inertia forces were assessed during the cyclic tests. Therefore, the hybrid seismic tests performed in this study aim to quantify and understand the true seismic behaviour of CFS moment resisting frames. The mechanism of energy dissipation and levels of ductility are of considerable importance in understanding how to design CFS moment resisting frame structures to withstand earthquakes.

The hybrid test method used in this paper has developed over the past 30 years ((Hakuno et al., (1969) & Takanashai et al., (1975)) to become a standard method of dynamic structural testing. The fundamental concept behind hybrid testing is that the portion of the structure that is difficult to numerically model e.g. highly nonlinear response, is physically tested using hydraulic actuators (McCrum & Broderick (2013)). This portion of the structure is referred to as the physical substructure. The remainder of the structure that is more easily numerically modelled e.g. only slightly nonlinear response, is numerically modelled at the same time as the physical substructure is tested. The numerically modelled portion of the structure is

referred to as the numerical substructure. The results (displacement & force) from the physical test feed into and update the numerical model in real time. The transfer of the physical test results into the numerical substructure results in the overall dynamic response of the structure being more accurately captured than a numerical model alone. Another great benefit is fewer laboratory facilities are required. Hybrid tests can be performed at slow loading rates (Pseudo Dynamic Testing (Shing & Mahin (1985, 1987), Takanashi & Nakashima (1987))) or close to real-time (Real-time Hybrid Testing (RTHT) (Nakashima et al. (1992))). In this project, soft-RHTs are performed and the methodology is discussed in more detail later in this paper.

A full-scale 2.2m high by 3.2m long moment resisting frame with three back-to-back section sizes; 203x76x20mm, 254x76x20mm and 300x95x20mm were seismically tested in this study. The full-scale portal frame tests performed in this study are essential as they will allow the full earthquake response to be captured and observed. Test results have shown the frames to have a stable hysteretic response, low levels of yielding at 1.53g peak ground acceleration and acceptable levels of interstorey drift. At very high peak ground acceleration local buckling of some of the column sections was observed at the column to haunch connection. This failure mechanism does not align with the strong column, weak beam design philosophy.

2. Moment resisting frame details

In terms of the design of CFS portal frames under dynamic loads, designers are given little normative guidance and must rely on limited test data. Therefore, design of CFS portal frames mainly relies on static load design guides. Dubina, et al. (2012) in a recent design manual present worked examples on the design of structural members in a CFS portal frame subjected to static loads. The manual does not offer guidance on the joint design in terms of strength and stiffness and design bending moments are calculated on rigid joint assumption. In this paper, however, the strength prediction of the joint moment resisting capacity is based on the local buckling of the channel web due to combined bending and bi-moment. The joint rotational stiffness was modelled as bi-linear spring using a maximum 3mm slip of bolts in tolerance holes (Wrzesien (2016)). The Elastic Method was used for predicting rotational stiffness of the bolt-group in the bearing phase of loading. The shear stiffness of the lap connection was calculated using analytical formula after Zadanfarrokh & Bryan (1992) and offered a conservative prediction of the maximum sway under monotonic load.

Three different back-to-back channel section portal frames were investigated in this study. Exact dimensions of the channel sections are as follows; 203x76x20mm with 1.8mm plate thickness (classified as C20018), 254x76x20mm with 2.0mm plate thickness (classified as C25020) and 300x95x20mm with 2.5mm plate thickness (classified as C30020). A schematic of the C25020 portal frame, connections and photographs of the connection details are shown in Figures 1, 2 & 3, respectively. The bracket thickness for all the haunch connections is 3mm. The grade of steel coil used for manufacturing sections is S450GD+Z275 according to (BS EN 10346:2015 2015) with nominal properties of $f_y = 450 \text{ N/mm}^2$ (0.2% proof strength), $f_u = 510 \text{ N/mm}^2$ (ultimate tensile strength) and 14% non-proportional elongation at maximum force (80mm gauge length). All bolts are M18 Grade 8.8.

3. Experimental set-up and hybrid test method

3.1 Experimental Set-up

The experimental set-up consists of a CFS portal frame sandwiched between two back-to-back strong reaction frames as shown in Figure 4. Support beams span between the two reaction frames as shown in Figure 4(a). The test frame is then supported off these beams using universal column stub sections. The stub sections have a small rotational stiffness and therefore replicate pinned connections. The dynamic actuator is mounted in between the two reaction frames as can be seen in Figures 4(a) & (b). During each experiment, force and displacement of the actuator head are recorded on an actuator Controller PC. In order to prevent out-of-plane displacement of the test frame during the experiments, two roller guides hold the test frame in place whilst allowing the test structure to move without friction in plane. Lateral in-plane displacement was measured by an internal LVDT in the actuator.

3.2 Hybrid Testing

In a soft-RTHT, the physical substructure and numerical substructure are combined using OpenFresco (Open Framework for Experimental Setup and Control) (Schellenberg (2009)). The data communication during a soft-RTHT is shown in Figure 5. The hybrid testing facility at Trinity College Dublin comprises of an MTS real-time hybrid test system. The hardware consists of a Series 111 MTS Accumulator, a 150kN capacity high speed linear hydraulic actuator with a 250mm (± 125 mm) stroke. The computer hardware comprises of; a Simulation Host PC, a Real-time Target PC, a Test PC and a two-channel MTS 493 Real-time Controller. The Structural Test System (STS) software provides the user interface for PID control and calibration of the actuator.

During a soft-RTHT, OpenSees (McKenna et al., (2000)) is used to create the dynamic model of the numerical substructure. The structural model is created on the Simulation PC and then downloaded onto the Target PC through a fibre optic cable (McCrum & Broderick (2013)). The sole task of the Target PC is to run the model in real-time (if necessary) using Mathworks xPC Target. As shown in Figure 5, the Target PC sends command displacements to the STS controller via the shared reflective memory called SCRAMNet GT150 (Shared Common Random Access Memory Network) (McCrum & Broderick (2013)). OpenFresco is the middleware that allows the communication between the finite element software (OpenSees) and the experimental hardware (MTS actuator). The Test PC provides the user interface to the Servo-controller and control of the actuator. As can be seen in Figure 5, command displacements (commDisp) are sent from the Servo-controller to the actuator. The measured force (measFrc) and displacement are sent back to the Servo-controller, then to the Target PC and finally the Simulation PC. In the Simulation PC the data is used to calculate the next time step command displacement, making the process close-looped (McCrum & Broderick (2013)).

The three-dimensional portal frame structure was modelled in OpenSees as shown in Figure 6. The lumped masses, P in Figure 6 were modelled as 108.5kg for all specimens as they had the same permanent and imposed loads from the spans above. In order to simplify the structure analysed, only two CFS portal frame bays were modelled. The structure is divided into physical and numerical substructures as shown in Figure 6. The masses were all numerically modelled at Nodes 7, 8, 11 & 12 in Figure 6. Therefore, the masses were numerical in the physical test. The beam members were all modelled as elastic elements with equivalent Young's Modulus, cross-sectional area, second moment of area (major & minor

axis) and torsional stiffness. The column members were explicitly modelled as fibre beam column elements with fiber cross sections and the relevant CFS material properties mentioned in Section 2. The lumped masses were all numerically modelled and the column fixity was pinned at the base and fixed at the top of the frame. The haunch connection was modelled as a rigid (fixed against rotation) connection between the column and beam elements in order to simplify the analysis and ensure computational efficiency. This may not truly capture the combined bending and bi-moment effect. Linear co-ordinate transformation was used for the beam members as it was assumed the damage would form in the columns and not the beams. In linear co-ordinate transformation a linear geometric transformation of beam stiffness and resisting force is performed from the basic system to the global-coordinate system (Mizzoni et al., (2006)). P-delta effects were included for the column members. The P-Delta co-ordinate transformation performs a linear geometric transformation of beam stiffness and resisting force from the basic system to the global coordinate system, considering second-order P-Delta effects (Mizzoni et al., (2006)). The P-Delta effect considers second-order moments resulting from the lumped mass (gravity load) being laterally displaced from the equilibrium position of the column member. The time history analysis was performed using BandGeneral system of equations, plain degree of freedom numbered, a Newmark integration scheme with fixed number of iterations and a Newton solution algorithm. Mass proportional Rayleigh damping was used. The physical substructure was modelled in OpenFresco in the same way as per McCrum & Broderick (2013) in order to use the laboratory resources as efficiently as possible.

3.3 Experimental Programme

The Taiwan 1986 earthquake record (see Figure 7) was specifically chosen for the hybrid tests from seven earthquake records with low, medium and high acceleration to velocity ratios (Calitri Irpinia 11/23/1980, Imperial Valley - El Centro 05/19/1940, Friuli Italy 05/06/1976, Kobe 01/16/1995, NW California 10/08/1951, Spitak Armenia 12/07/1988). From these records, the Taiwan record had the largest and most stable simulated displacement response after numerical simulations were performed. The natural period of the portal frame structure was numerically simulated from the OpenSees model as 0.76s. The lumped masses on this one-storey CFS portal frame structure shown in Figure 6 are low and therefore the inertia forces and subsequent roof displacements are relatively low. CFS structures can sustain large lateral in-plane deflection without failure and therefore the stroke of the actuator would be a limiting factor in achieving collapse/ultimate failure. The lateral in plane yield displacement of the frames was calculated using ECCS (1986) method (see McCrum et al., (2018)), and on average for the frames was; C20018 at 21.5mm; C25020 at 16.8mm and C30025 at 19.4mm. Therefore, the selected ground acceleration record used in this study was scaled by factors of 1.0, 3.0 and 5.0, respectively, in order to achieve adequate lateral in-plane frame deflections in the nonlinear response. The scaling for each of the hybrid tests is shown in Table 1. Each block of tests e.g. Test 1, Test 2 and Test 3 were performed all on the same C20020 portal frame specimen. Therefore, the results for Test 3 represent the cumulative damage from Test 1 and Test 2. Test 2 was accidentally scaled to 2.0 rather than 3.0 as per the other tests. It was not possible to scale Test 9 to 5.0 as numerical issues prevented the hybrid test from running. Therefore Test 9 was run at 4.0 scale.

4. Experimental results

The results in Table 2 show the initial stiffness, total energy dissipated, peak lateral in plane frame displacement/force and peak interstorey drift for all of the tests. Figure 8 shows the hysteretic response of Tests 1, 4 and 7.

From Table 2 it can be seen that the C20018, C25020 and C30025 specimens had interstorey drifts of 0.66%, 0.53% and 0.55%, respectively for a 1.0 scaled excitation ($PGA = 1.53g$).

Assuming the Importance Class for single-storey CFS portal frames is Class I or II, then the reduction factor, v is recommended as 0.5 in Eurocode 8 (BS EN 1998-1 (2004)). Even with the lowest limit of 1% for interstorey drift for brittle non-structural elements the three frames performed well having interstorey drifts well below 1%. Visual observation during and after Tests 1, 4 & 7 showed no signs of local buckling in any part of the structure. The test structures exhibited stable hysteretic responses when subjected to a 1.0 scaled excitation ($PGA = 1.53g$) as shown in Figure 8. There is some nonlinearity in the hysteresis in Figure 8(c) for the C30025 section frame. There is almost no nonlinearity in the C25020 section specimen in Figure 8(b). There is no nonlinearity in the C20018 section frame in Figure 8(a). At first glance, the results that nonlinearity is observed in the stiffest frame (Figure 8(c)) and not in the most flexible frame (Figure 8(a)) may seem unusual, however the change in stiffness of the structure results in a change in spectral response (refer to Figure 7(b)). In Figure 8(a) the period is 0.37s resulting in a spectrum acceleration of approximately 0.25g, whereas in Figure 8(c) the period is 0.75s and the spectrum acceleration is approximately 0.5g. Therefore, the stiffer frame is subjected to a higher acceleration.

Figure 9 shows the hysteretic response of Tests 3, 6 and 9, respectively. There is only a slightly nonlinear response in Test 3 (Figure 9(a)) for the C20018 frame. No local buckling was observed during or after the test. The reduction in stiffness of the C25020 section frame is evident in the hysteresis in Figure 9(b). In Test 6, the C25020 specimen locally buckled in the column web and flanges at the haunch to column connection. In Figure 9(c) an almost isotropic hardening and stiffness degradation for the C30025 specimen. In Test 9, the C25020 specimen locally buckled in the column web and flanges at the haunch to column connection. The structures perform well considering the cumulative loading and extreme loading conditions the specimens have been subjected to.

Figure 10 shows the strain gauge plots for Tests 7, 8 and 9 and Figure 10 shows the strain gauge locations (Strain Gauge #6 is the strain gauge results in Figure 9). The yield strain is approximately 2×10^{-3} . It can be seen in Figure 10 that the test specimen only yields in Test 9 where the peak ground acceleration was 6.12g. The strain gauge data matches the observations from the experiment in that Test 7 and Test 8 showed no visible signs of local buckling. In Test 7, the strain results from Strain Gauge #6 (refer to Figure 11 for the location of strain gauges) show no nonlinearity in the material, so it is suspected that the nonlinearity in the hysteresis plots in Figure 8(c) result from slippage in the bolt group and friction between the haunch plate and beam/column sections. This may be larger for the larger sections with greater surface area between the haunches and beam/column sections. The video still images of the left-hand side column/haunch connection in Test 9, as shown in Figure 11, capture the initiation of local buckling in Test 9 in the pull (Figure 11(a)) and push (Figure 11(b)) loading direction.

The test results demonstrate that the test specimens performed well during the real earthquake loading. The local buckling in Test 9 reduced the lateral stiffness of the specimen as can be seen in the hysteresis in Figure 9(c). It should be noted that the local buckling in the column sections at the column to haunch connection would reduce the axial load capacity of the

column members. This was also observed in similar cyclic tests (McCrum et al., (2018)). The plastic hinge location would form at the joint between the column to haunch connection rather than in the beam to haunch connection. The preferable failure mechanism is in the beam i.e. strong column, weak beam design philosophy.

Aside from the failure mechanism, the three portal frames subjected to three subsequent and increasing peak ground acceleration scaled earthquakes performed very well. The CFS portal frames perform well, however the failure mechanism needs to be changed to produce a structure that can continue to carry axial load after an earthquake event.

5. Conclusions

Nine scaled real earthquake tests on three cold-formed steel portal frame structures was undertaken using the hybrid test method in this study. The portal frames were composed of back-to-back channel sections 200mm x 76mm, 250mm x 76mm and 300mm x 95mm. The results showed that when scaled to 1.53g, the frames did not yield and performed well with interstorey drifts well below allowable limits. A stable hysteretic response was observed at much higher levels of peak ground acceleration and good levels of energy dissipation were observed. The 200x76 section did not yield or locally buckle up to a PGA of 7.65g. The 250mm x 76mm and 300mm x 95mm moment resisting frames locally buckled and yielded at earthquake excitations of 7.65g and 6.12g, respectively. The local buckling in these tests occurred at the column to haunch connection. Such local buckling in the columns would reduce the lateral stiffness of the frames and the axial load capacity of the column members. Overall, the CFS portal frames perform very well, however the failure mechanism needs to be altered from a designer's perspective to produce a structure that can continue to carry gravity loads after an earthquake event. The preferable failure mechanism is in the beam i.e. strong column, weak beam design philosophy.

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List of notation

- f_y is the 0.2% proof strength of the steel
 f_{ult} is the ultimate tensile strength of the steel

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Table captions

Table 1. Experimental programme for hybrid tests

Table 2. Summary of hybrid test results

Table 1. Experimental programme for hybrid tests

Test No.	Test type	Connection Member	Earthquake Record	Scaling
1	C20018	FHB200B	Taiwan	1.0
2	C20018	FHB200B	Taiwan	2.0
3	C20018	FHB200B	Taiwan	5.0
4	C25020	FHB200B	Taiwan	1.0
5	C25020	FHB200B	Taiwan	3.0
6	C25020	FHB200B	Taiwan	5.0
7	C30025	FHB300B	Taiwan	1.0
8	C30025	FHB300B	Taiwan	3.0
9	C30025	FHB300B	Taiwan	4.0

Table 2. Summary of hybrid test results

Test No.	Section Size	Scaling	PGA (g)	Initial Stiffness (N/mm)	Total Energy Dissipated (J)	Peak Lateral Disp. (mm)	Peak Interstorey Drift (%)	Peak Lateral Force (kN)
1	C200	1.	1.5	11	59	13.	0.66	12.
2	18	0	3	55	610	29	2.25	65
3	C20018	2.0	3.06	1033	1688	44.92	3.32	31.74
	C20018	5.0	7.65	939		66.34		28.87
4	C250	1.	1.5	25	397	10.	0.53	19.
5	20	0	3	82	4173	55	1.51	59
6	C25020	3.0	4.59	2526	12452	30.26	2.54	47.18
	C25020	5.0	7.65	2155		50.87		58.83
7	C300	1.	1.5	48	1934	10.	0.55	37.
8	25	0	3	31	21173	96	1.68	17
9	C30025	3.0	4.59	4404	38037	33.63	2.28	89.26
	C30025	4.0	6.12	4300		45.64		107.17

Figure captions

Figure 1. Schematic of C25020 portal frame (McCrum et al., (2018))

Figure 2. Schematic of beam to column haunch connection details for (a) C20018 sections;
(b) C25020 sections; and (c) C30020 sections

Figure 3. Photograph of; (a) beam/column haunch connection; and (b) base plate connection
for C25020 test specimen

Figure 4. (a) Schematic of the experimental set-up; and (b) photograph of experimental set-up
for C300 specimen showing back-by-back reaction frames with CFS portal frame and
actuator inset in between (McCrum et al., (2018))

Figure 5. Schematic of hardware and data communication for the soft-real time hybrid test

Figure 6. Schematic of physical and numerical substructure

Figure 7. 1986 Taiwan earthquake record; (a) acceleration versus time (unscaled); and (b)
pseudo acceleration versus period response spectrum (PEER (2015))

Figure 8. Hysteresis plots (actuator force vs lateral in-plane displacement) for hybrid tests of
CFS portal frame; (a) Test 1; (b) Test 4; and (c) Test 7

Figure 9. Hysteresis plots (actuator force vs lateral in-plane displacement) for hybrid tests of
CFS portal frame; (a) Test 3; (b) Test 6; and (c) Test 9

Figure 10. Plot of strain versus time for; Test 7 (top); Test 8 (middle) and; Test 9 (bottom) for
Strain Gauge #6

Figure 11. Close-up photographs during Test 9 indicating; (a) initial local buckling in the web
and flange on the pull loading from the actuator; (b) initial local buckling in the flange
on the push load directly after (a); and (c) location of strain gauges on cross section

Figure 1

Figure 2

(b)

(c)

Figure 3

Figure 4

Figure 5

Figure 6

Figure 7

Figure 8

Figure 9

Figure 10

Figure 11