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Abstract

A series of monotonous loading and hysteresis loading tests on cold-formed steel portal frames were conducted in this paper. The averaged ductility factor value of tested frames is 3.15 and the strength and stiffness degradation are not obvious during the test. The failure mode of frame is local buckling at column bases followed by local buckling at the top of columns, which lead to the dropping of frame's load-carrying capacity. Then, the finite element model is developed and the analysis results match well with the test results. The research in this paper indicates that cold-formed steel portal frame has a good seismic performance.

Introduction

Cold-formed steel portal frame is consisted of cold-formed steel beams and columns, which are connected with connection element by self-drilling screws or high-strength bolts (Lu et al. 2008). Cold-formed steel portal frame structure has been widely used in agricultural buildings, small and medium-sized commercial buildings in Australia, the United States, Japan and other developed countries. At present, domestic and foreign researches are mainly focused on the connection strength and integral frame's ultimate load-carrying capacity, while the study on the seismic performance of portal frame is relatively less (Wrzesien and Lim 2008).

In this paper, a series of monotonous loading and hysteresis loading tests on cold-formed steel portal frames were conducted, and frame's failure mode and ductility were discussed to assess this kind of structure's seismic performance.

Experimental investigation

Test setup

Six realistic portal frames, composed of single lipped channel sections or back-to-back lipped channel sections, were tested in this research, and they were classified into three types (frame A, B, and C) based on cross-section geometry (as shown in Fig. 1 and Table 1). For each type of

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frame, one is monotonously loaded and the other is cyclically loaded. Portal frame's dimension is shown in Fig. 2.

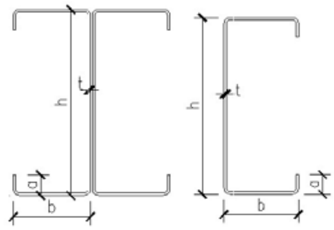


Fig. 1 Cross-section geometry

Table 1 Nominal cross-section dimensions

Frame	*h*(mm)	*b*(mm)	*a* (mm)	*t* (mm)
A	150	64	15.5	1.5
B	150	64	15.5	1.5
C	200	76	15.5	2.0

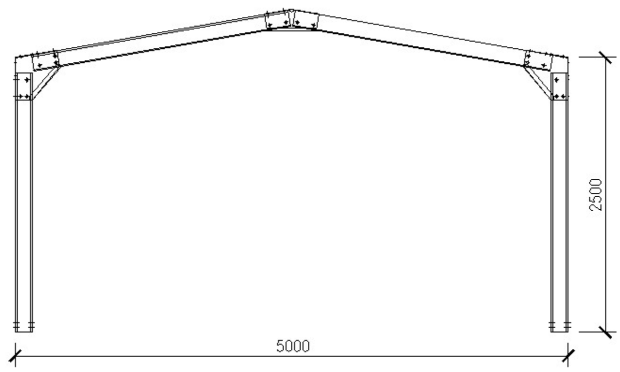


Fig. 2 Portal frame dimensions

Material properties

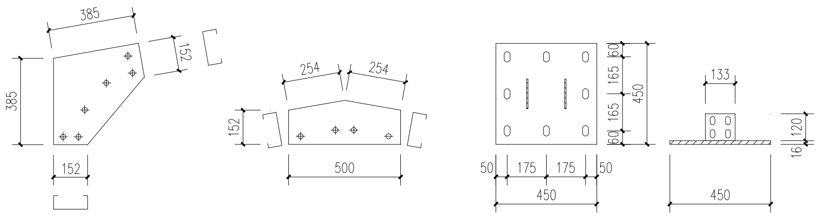
The structural steel grade of the cold-formed steel section was Q345 with nominal yield strength of 345MPa. Four kinds of plate thickness were used in the tests. The material properties including yield stress (F_u) and elastic modulus (E) for each thickness were obtained through coupon tensile test and the test results are summarized in Table 2.

Table 2 Material properties

Thickness (mm)	F_y (MPa)	E ($\times 10^5$MPa)
1.5	387	2.02
2.0	489	2.06
2.5	406	2.03
6.0	404	2.01

Joints

Joints are an important part for portal frames. The details of the joints, including beam-column joints, beam-beam joints, and column base joints, are shown in Fig. 3. For all joints in the tests, high-strength bolts in bearing type were adopted, whose pretension force was taken as 100kN to meet the requirement specified in "*Design of steel structures*"(GB50017-2003, 2003).



(a) Beam-column joint (b) Beam-beam joint (c) column base

Fig. 3 Joint details (taking frame B for example)

Loading and measuring systems

In this experiment investigation, column's axial compression ratio caused by representative gravity load was less than 0.1. Since the finite element analysis indicates that this vertical load has little impact on frame's hysteretic behavior, only horizontal load was applied by a 10t actuator with displacement control during the test.

In the hysteretic test, the loading procedure was consisted of two parts. In part one, when the horizontal displacement at beam-column joint was less than 20mm, four loading cycles at 5mm, 10mm, 15mm, and 20mm were adopted to determine the "yield load" and "yield displacement". In part two, several loading cycles were adopted at 30mm, 40mm, 50mm, etc. until the applied load dropped to about 85% of the peak load. Each loading cycle in part 2 repeated for two times.

Totally 38 strain gauges were arranged on each frame to detect the stress distribution on some "key-parts", such as beam-column joints and column bases. The LVDTs were used to record the horizontal displacement at beam-column joints and rotation angles at connections. It should be noted that the rotation angle was calculated by relative displacement at each connection.

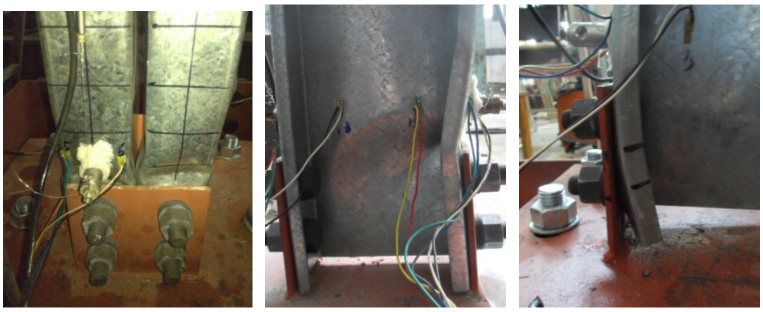
Test results analysis

Test phenomenon

Monotonous test and hysteresis test of portal frame A and B basically demonstrated the same failure characteristics. Local bucking firstly happened at both column bases and then the first plastic hinge appeared there (as shown in Fig. 4 and Fig. 5) , leading to the decline of the portal frame's lateral stiffness. In the monotonous test, plastic hinges appeared at column bases followed by the formation of plastic hinge at the end of beam near loading location. In the hysteresis test, after two plastic hinges appeared at the column bases, local bucking occurred at the top of the column located at the loading end when the portal frame was subjected to pulling, resulting in the decline of the portal frame's capacity.

The failure characteristic of portal frame C was different from portal frame A and B. The column base displayed deformation because the flexural rigidity of portal frame C was larger. In the monotonic test, the bolts at the column bases overcame friction and squeezed with the bolt holes at the bottom of columns, causing deformation at bolt holes. In the hysteresis test, tilted deformation happened at the bezel of column bases, located near the compressive side of the bottom of columns. And the above reasons made the rotational stiffness of column bases decline, reducing the lateral stiffness of the portal frame.

The relative rotation can be neglected at beam-column joints and beam-beam joints based on measured displacement. According to the conclusion of literatures(Lim and Nethercot 2004a; Lim and Nethercot 2004b)， beam-column joints and beam-beam joints of portal frames can be regarded as rigid joints. The relative rotation angle at column base is much larger than rotation angles at beam-column joints and beam-beam joints, showing a certain degree of semi-rigid joints characteristics.



(a) Frame A (b) Frame B (c) Frame C

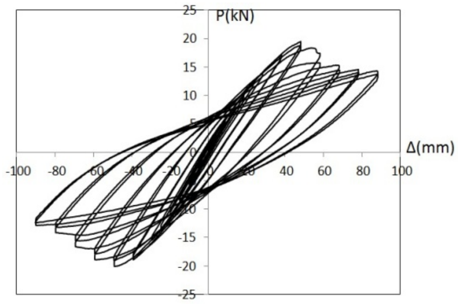
Fig. 4 Failure modes at column bases in monotonous tests

(a) Frame A (b) Frame B (c) Frame C

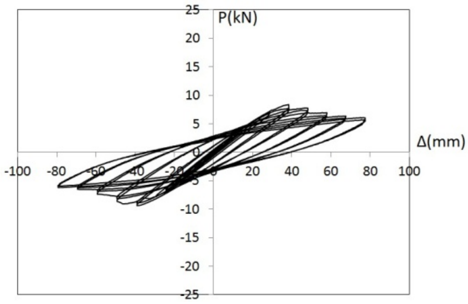
Fig. 5 Failure modes at column bases in hysteresis tests

Load versus displacement curves in cyclic tests

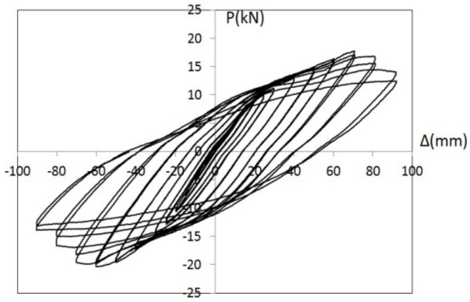
The load versus displacement (P-Δ) curves are presented in Fig. 6, in which Δ is the averaged horizontal displacement at the top of columns. As shown in Fig. 6, the hysteresis curves of all portal frames in the test are plump, indicating cold-formed steel portal frame structure has good plasticity and hysteretic energy-dissipation capacity. In general, the cold-formed steel portal frame structure displays good seismic performance.



(a) Frame A



(b) Frame B

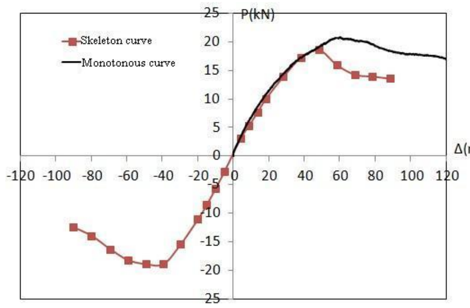


(c) Frame C

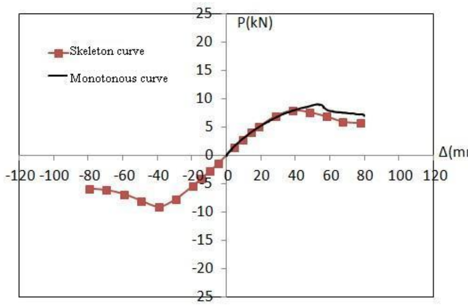
Fig. 6 Hysteresis curves

Skeleton curve and load versus displacement curve in monotonic test

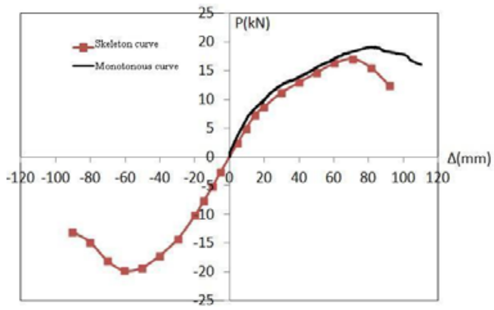
The comparison between skeleton curves obtained from hysteresis curves and load versus horizontal displacement curves obtained by cyclic test is summarized in Fig. 7. The stiffness of skeleton curve is very close to P-*Δ* curve in the monotonic test, and frame's lateral stiffness and ultimate load-carrying capacity have no significant drop under cyclic load. As shown in Fig. 7, the skeleton curve is not completely symmetrical due to Bauchinger effect and unsymmetrical constraints of frame during pushing and pulling process.



(a) Frame A



(b) Frame B



(c) Frame C

Fig. 7 Skeleton curves and monotonic curves

Ductility analysis

The ductility factor of the structure is an important indicator to evaluate the ductility. There are a variety of methods to determine ductility factor. In this paper, the energy method (Fan and Zhuo 2001) is adopted to calculate ductility factor. As shown in Fig. 8, the yield deformation (Δ_y) and yield load (P_y) can be determined by assuming area A_1=A_2. The ductility factor (u) shall be determined in accordance with equation (1)

$$u = \Delta_u / \Delta_y \quad (1)$$

where Δ_u is ultimate deformation corresponding to ultimate load P_u, and Δ_y is yield deformation. All frames' ductility factors are reported in Table 3.

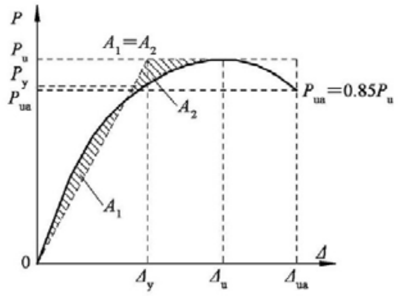


Fig. 8 Calculation of ductility factor using conservation of energy method (Xu et al. 2010)

Table 3 Ductility factors of portal frames

Frame	Loading direction	Δ_y (mm)	Δ_u (mm)	u
A	Positive direction	22.2	68.5	3.0
	Opposite direction	-24.1	73.8	3.1
B	Positive direction	18.5	60.2	3.3
	Opposite direction	-20.1	60.9	3.0
C	Positive direction	23.6	82.0	3.5
	Opposite direction	-25.2	-76.8	3.0

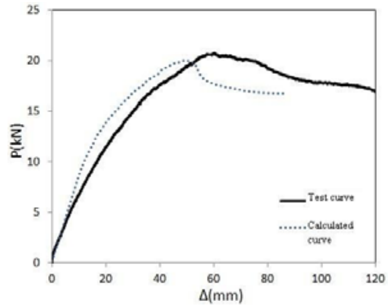
Finite element analysis

Finite element model

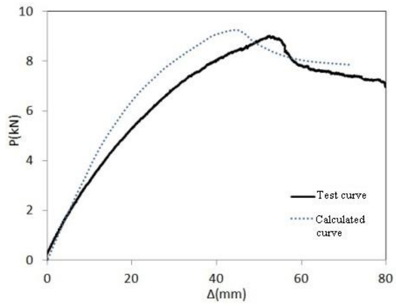
The finite element software ANSYS was used to simulate and analyze a single-bay frame. The beams, columns and gusset plates were modeled by SHELL181, which is a 4-node element with six degrees of freedom at each node. The bolted connection of the frame was achieved by coupling nodes. The bilinear kinematic hardening material model is employed in ANSYS model, in which the material properties including yield stress (F_y) and elastic modulus (E) were from coupon tensile test. The Poisson's ratio was taken as 0.3 and the tangent modulus is set to 0.02E.

Results comparison between finite element analysis and test results

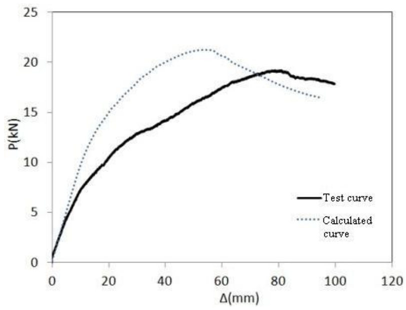
The comparison of load versus displacement curves in monotonous loading type between finite element analysis and test is shown in Fig. 9, and the comparison of hysteresis curves is shown in Fig. 10. The finite element analysis results match well with test results except for frame C. This disagreement is partly caused by the fact that finite element method cannot simulate the bolt slip at column base, which was observed during the test of frame C.



(a) Frame A

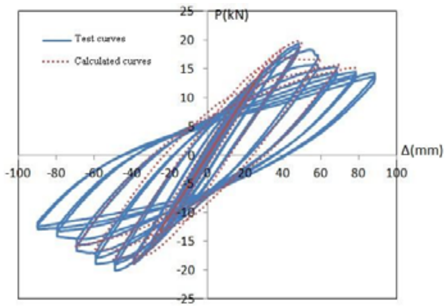


(b) Frame B

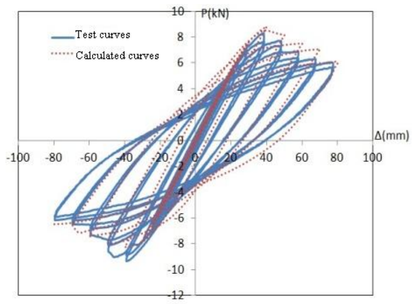


(c) Frame C

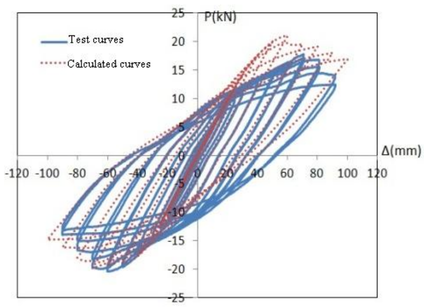
Fig. 9 Comparison between finite element analysis and test results in monotonous test



(a) Frame A



(b) Frame B



(c) Frame C

Fig. 10 Comparison between finite element analysis and test results in cyclic test

Conclusions

(1) The hysteretic loops of cold-formed steel portal frames are plump, and the pinch effect is not obvious. The test results indicate that the portal frame has a good plasticity and hysteretic energy-dissipation capacity, which shows great seismic performance.

(2) Failure mode of cold-formed steel portal frame structure under monotonic load is slightly different from it under cyclic load. In the

monotonic test, local buckling firstly occurs at column base and then occurs at the end of beams opposite to loading location. In the cyclic test, local buckling occurs at column base followed by local buckling at the top of column near the loading location. The failure of connection does not appear throughout the test, which meets the seismic design requirement of “strong joints and weak members”.

(3) The finite element analysis results are in good agreement with experimental results, indicating that finite element method can well predict the structural behaviors of cold-formed steel portal frame if high-strength bolts at column base do not slip.

Acknowledgements

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