Lateral Torsional Instability of Single Channels Restrained by Angle Cleats

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Twentieth International Specialty Conference on Cold-Formed Steel Structures
St. Louis, Missouri, U.S.A., November 3 & 4, 2010

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Abstract

A series of experiments on the lateral torsional instability of single channels is presented. The channels are restrained by a purlin – angle cleat connection and subjected to a two point loading system in order to simulate a distributed load. Failure of the channels occurred by local buckling of the compression zone of the flange and web and lateral torsional buckling of the channels between points of lateral support. Tests have shown the purlin–angle cleat connection to be capable of restraining the frames from failing due to lateral-torsional buckling. This eliminates the idea of having fly-bracings, which is normally done in practice to restrain torsional instability.

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Introduction

Cold-formed channel beams and rafters in portal frames are usually restrained against lateral-torsional buckling behaviour from its top flange through an angle cleat-purlin connection. Additional restraint is usually provided by fly bracing. This restraining mode has disadvantages of either weakening the top flange of the main frames if it is in tension, due to tearing that occurs around the fastened points or fabrication costs of providing fly bracing. This study investigates a restraint that avoids the use of fly bracing and bolt holes in the top flange. Restraint of the beam or main frame is still provided by a purlin-angle cleat connection, however the angle is long enough so as to connect the main beam or frame in the web. This connection configuration is found to be better because it restrains both lateral and torsional movements of the member.

In these tests the beam, purlin and angle cleats are all cold-formed steel sections to make the structure light and connected together by bolting only. The sizes of the beam, purlin and angle cleat section are 300x75x20x3mm, 100x50x20x2mm and 100x75x20x3mm, respectively. In order to obtain different buckling modes, the length of the channels is varied from 1.8 to 6m. The support systems were designed to achieve simply supported end conditions in the vertical plane; however the channel beams were restrained against lateral deflections and twist rotations at the ends. Restraints were also provided at the loading points. The beans were subjected to a two-point loading system at the top flange in order to experience pure bending in the internal span. The objectives of the tests are to examine the ability of thin cold-formed angle cleat to restrict lateral-torsional buckling and compare the test results with unfactored resistances from design standards.

Material and section properties

The channel sections used are of commercial quality steel. A total of fifteen coupon test specimens were cut from the web and flange of channel beams. Corner coupons were not tested because of the lack of appropriate tools to prepare and test them. The coupon were prepared and tested in a 100kN capacity displacement controlled testing machine according to the guidelines provided by the British Standard, BS 18. The thickness and width of the reduced section of coupons were measured and recorded on the computer system so as to calculate the area and subsequently the stresses. The longitudinal strain gauges, attached to the coupon at the centre of each face, were used to determine the strains. The tensile load was applied to the prepared coupon test at a constant rate of 3.0mm/min until failure. A 50mm gauge length was marked onto the tensile test specimens before testing. After fracturing the specimens the two parts are fitted together to measure the axial elongation of the coupons. The ductility of the steel is evaluated as a percentage of the elongation at failure, according to the following equation:

$$\varepsilon_f = \left(\frac{l_f - l_0}{l_0} \right) \times 100$$

where, l_0 is the initial length of gauge and l_f the final length measured after fracture.

The stress-strain relationship of the coupons, shown in Figure 1, is derived from the load-elongation relationship using its original cross-sectional area and the gauge length. The yield stress, ultimate stress and modulus of elasticity of the steel are determined from these stress-strain curves. The average yield stress and tensile stress of the web and flange coupons are summarized in Table 1. In this table, ε_y and ε_u are the yield and the ultimate strain respectively. In compliance with SANS-10162-1:2005, the material properties of the channels achieved the recommended ductility requirements, that is, the percentage elongation at failure exceeded 10% for a 50mm gauge length and the ratio of the specified ultimate tensile strength (f_u) to the specified yield strength (f_y) exceeded 1.08.

The measured dimensions for the channels under investigation were found to be very close to the nominal ones from the supplier. This allowed the use of section properties from the Southern Africa Steel Construction Handbook (2005).

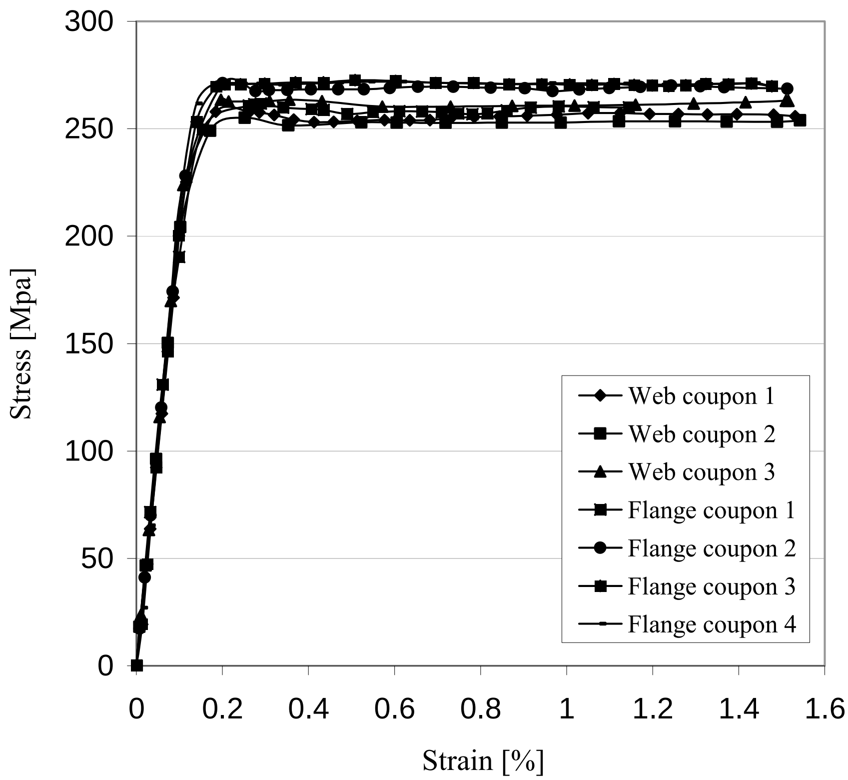

Figure 1 Stress-Strain Curve

Table 1 Average material properties of the channels

Specimen	f_y (MPa)	f_u (MPa)	f_u/f_y	ε_y	ε_u	$\varepsilon_u/\ \varepsilon_y$	ε_f (%)
Web	259.17	367.62	1.42	0.015	0.028	1.83	41.75
Flange	273.66	375.36	1.37	0.022	0.034	1.58	42.45

Test programme

Nine beams were tested under two point loading as illustrated by the schematic diagram in Figure 2. Two-point loading provides a constant moment region between the applied loads so that pure bending failure only is experienced. This loading arrangement simulates a distributed load over the entire span of the beam.

The span of the beams varied from 1.8m to 6 m. The support system was designed to ensure that the beam test is simply supported and, that both twisting and lateral deflections were prevented at the ends. Lateral and torsional bracing was also provided at the loading point as shown in the figure. Details of the span and points of bracing are shown in Table 2. The load was applied through the top flange of the channel sections, exactly at the restrained points to simulate the tests to the actual conditions to which a frame could be subjected under vertical load.

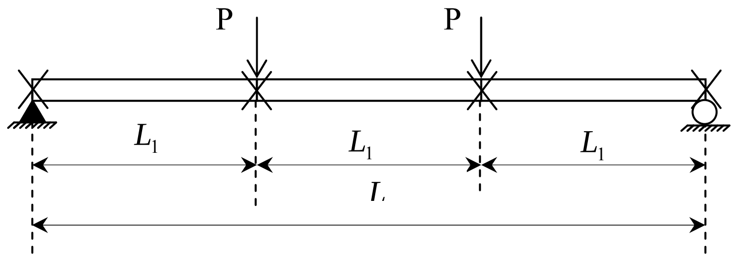

Figure 2 Sketch of test set-up

Table 2 Length of tested beams

Tests	Span, L (mm)	Length L_1 (mm)	Slenderness ratio of internal length
Test 1	1 800	600	23.9
Test 2	2 280	760	30.3
Test 3	2 790	930	37.1
Test 4	3 300	1 100	43.8
Test 5	3 780	1 260	50.2
Test 6	4 290	1 430	57.0
Test 7	4 800	1 600	63.7
Test 8	5 280	1 760	70.1
Test 9	6 000	2000	79.7

To allow for interaction to occur between members in an assembly, the beams were tested in pairs, as shown in Figure 3 (Baker and Eickhoff 1955, 1956; Baker et al. 1956; Dowling et al. 1982 and Dundu and Kemp 2006). The channels are oriented in the same direction as this offers greater stiffness than having the channels oriented in different directions (Dundu and Kemp 2006). The beam channels in each assembly are spaced at 1.84m, and as indicated before were connected together by 100x50x20x2mm cold-formed purlin sections through 100x75x20x3mm cold-formed angle-cleats. Two, 12mm diameter bolts connects the angle-cleat to the web of the purlin whilst another two, 20mm diameter bolts connects the same angle cleat to the web of beam channel.

Figure 3 Typical test set-up

The beams were fully instrumented so that in-plane deflection, out-of-plane deflection, strains and torsion rotation of the beam could be measured. These measurements were recorded at the mid-span through a data logger. In-plane and out-of-plane deflections of the beam were measured using 3 linear variable differential transducers (LVDTs) as shown in Figure 3. Torsional rotation at mid-span of the beams was monitored by means of clinometers placed inside the web. Strains were measured in both the top flange and bottom flange of the channel in order to determine the moment-curvature behaviour of the frames. A 250kN hydraulic instron testing machine was used to apply the loads. Each test specimen was incrementally loaded at the rate of 2mm/min until failure. All measurements were taken at each load increment, until the beam tests buckled.

Experimental results

The results of the full scale beam tests are summarised in Table 3. P and M_u are the maximum point load and moment applied to the base, respectively, and M_r is the buckling moment resistance, determined using the South African structural steel code, SANS10162-2:2005. This code is based on the Canadian structural steel code, CAN-S16.1-M89. As expected, small in-plane deflections and higher ultimate load were observed in short beam tests by comparison with longer beams. The buckling moment of resistance of the middle unbraced length is determined, based on modified section properties (effective width of compression elements) to control local buckling. The effective width concept was first proposed by Von Karman (1932) and calibrated for use by Winter (1947). Since the load was applied at the top of the channels it had a distabilising effect on the channels. This implies that an effective length factor of 1 for bending about the minor axis (assuming a partially restrained member) and moment-gradient factor of 1 (uniform bending moment diagram) should be adopted. A comparison of the experimental moment and the buckling moment resistance shows the experimental moment to be significantly lower than the buckling resistance. This is because the some of the spans used are not long enough to encourage a larger moment to develop. Additional tests that favour a lateral torsional buckling mode of failure are being pursued.

Table 3 Test results

Tests	Unbraced-length (mm)	P (kN))	M_u (kNm)	M_r (kNm)
Test 1	600	21.92	13.2	33.69
Test 2	760	18.51*	12.8	33.69
Test 3	930	17.98	16.9	33.69
Test 4	1 100	15.48	17.0	33.69
Test 5	1 260	14.50	18.41	33.69
Test 6	1 430	13.9	20.0	33.68
Test 7	1 600	13.03	20.8	32.8
Test 8	1 760	11.29*	20	31.9
Test 9	2000	11.36	22.7	30.45

* Instron stopped during testing

Local buckling failure of the compression flange-web junction was observed in all tested channel beams. This mode of failure occurred at the point where the load was applied and was probably caused by stress concentrations emanating from the load itself. Figure 4 shows the observed buckling mode. No evidence of lateral torsional buckling failure was witnessed, implying that the angle cleat-purling connection was able to control lateral torsional buckling.

Figure 4 Failure of the compression flange-web junction

Graphs of the relationship between load and displacement of frames are shown in Figure 5. The behaviour consists firstly of a linear response followed by a non-linear response. After this point large deformations take place and result in the collapse of the frame. As shown in this figure the buckling load decreases with increase in length of the channel. Note that the load and deflection in these graphs excludes the effect of the loading system.

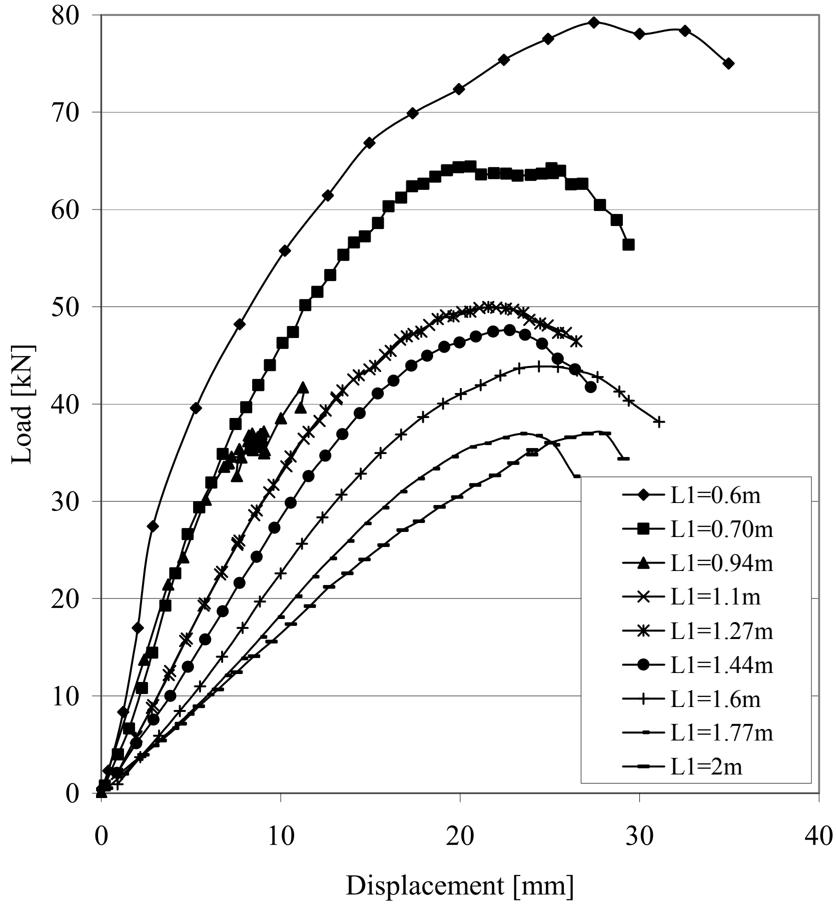

Figure 5 Load-Displacement curves

Conclusion

Tests on the lateral buckling of cold-formed channels beams under two- point loading have been described. The following conclusions are made:

- The lateral-buckling strength values obtained from the tests are in all cases less than the values predicted by the Canadian/South-Africa code of practice.

This is because some of the spans used are not long enough to encourage a larger moment to develop. Future tests are expected to correct this.

- In all cases failure occurred by local buckling of the compression flange-web junction. The capacity reached by the channels shows that a purlin-cleat restraining system is able to resist lateral buckling, and it can be used without adding fly bracing, as is normally done in practice to restrain torsion instability.

Acknowledgements

The authors wish to thank University of Johannesburg Research Committee (URC) for sponsoring this research.

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