USE OF ANGLE CLEATS TO RESTRAIN COLD-FORMED CHANNELS AGAINST LATERAL TORSIONAL INSTABILITY

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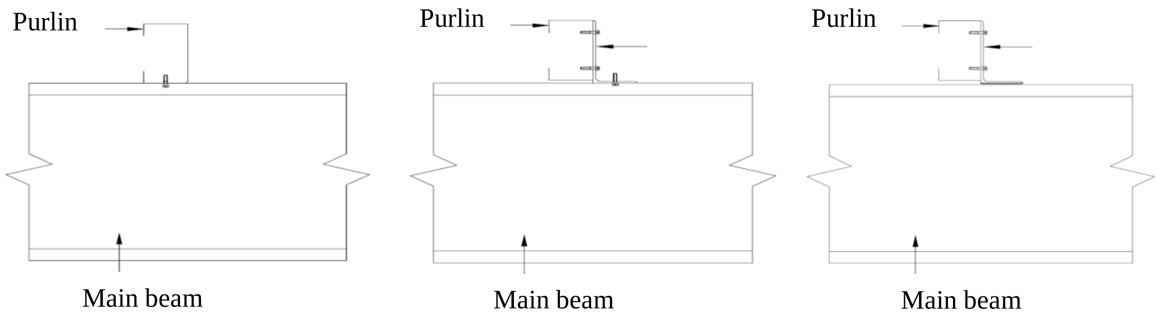
Abstract

It is common practice in the steel construction industry to restrain members that largely in flexure and torsion using a combination of angle cleats, connected at the top flange, and fly-bracings. This system is complicated and expensive, especially when used to restrain channels in bending. This paper investigates experimentally the use of angle cleats, connected to the webs of both the purlin and the channels, as a restraining system. Pairs of channels were subjected to a two point loading system in order to simulate a distributed load. Variable in the tests include the unbraced length between the two-point loads and the size of the channels. Failure of the channels occurred by lateral torsional buckling and catastrophic distortional buckling of the intermediate unbraced length. Tests showed that the purlin-cleat restraining system is able to resist lateral torsional buckling of the channels, and that this system can be used without any fly bracing. Distortional buckling was the final failure mode, and it occurred at moments less than the predicted lateral-torsional buckling moment of resistance. Distortional buckling is more critical in frames with shorter unbraced lengths and thicker channels.

Keywords: single channels, restrained, purlin–angle cleat, connection, lateral-torsional instability, catastrophic distortional buckling.

1. Introduction

Lipped cold-formed channels are among the most used thin sections in the steel construction industry. The demand for these structural elements has increased remarkably during the last decade, especially in residential, industrial and commercial buildings. In these structures, the smaller sections are normally used as purlins and diagonal bracing elements, and the larger sections are used as the main beam. When cold-formed steel lipped channels are used as the main beam members they are usually restrained against lateral buckling behaviour by purlins at the top flange. This restraining system works together with an additional restrain system, called fly-bracing, to prevent torsional instability. As shown in Figure 1(a) the purlin can be connected directly to the main beam or through an angle cleat, of the same width as the beam section, as shown in Figure 1 (b) and (c). In a common angle-cleat connection, one leg of the angle is connected to the web of the purlin through bolts and to the top of the main beam through either a bolted or a welded connection (Figure 1 (b) and (c)). The disadvantage of these restraining systems is that when the purlin/angle cleat is bolted or welded to the main beam, the bolt-hole or the welding process weakens the bearing length of the channel, especially when the purlin is subjected to large downward loads. Conversely the bolted area can easily tear-out if the top flange is in tension. In addition, the combined cost of providing this restraining system is high.



(a) Direct connection (b) Bolted angle cleat c) Welded angle cleat

Figure 1 Purlin-beam connections

Due to the above reasons, this study investigates the use of a restraining system that avoids bolt holes and welding in the top flange of the main beam, and the use of fly bracings. Restraint of the main beam is still provided by a purlin-angle cleat connection; however the angle is long enough to connect the webs of the purlin and the main beam. Details of the restraining system are shown in Figure 2. The restraining system consists of a lipped cold-formed angle cleat, connected to the main beam using 2, M20 mm diameter bolts, and connected to a purlin using 2, M12 mm diameter bolts. This means that the angle cleat restrains both lateral and torsional movements of the member. Since the angle cleat connects both elements (purlin and main beam) in the web, the proposed restraint has the added advantage of preventing the main beam and purlin's web from crippling at loading points. Each hole is located at 35mm from the top and bottom flanges to take advantage of the increased stiffness close to the corners of the channels.

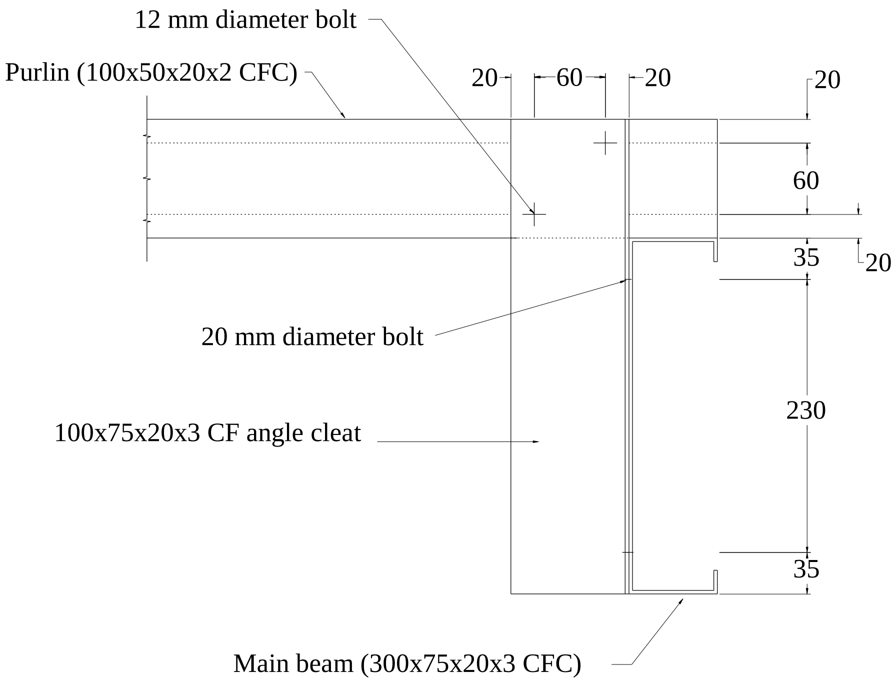


Figure 2 Typical purlin-beam connection

The proposed restraining system has been used in portal framed structural systems, in previous investigations [1, 2, 3], and was found to be efficient in restraining lateral-torsional instability.

In this study, three possible modes of failure were observed in the portal frames tested, namely: local buckling of the compression zone of the flange and web of the channels, lateral-torsional buckling of the channels between points of lateral support, and bolts in bearing. However, the governing failure mode in all these frames was not the lateral-torsional buckling failure mode. This means that the restraining capacity of the angle cleats could not be sufficiently ascertained. After considerable relative rotation of the channel sections within the eaves connection, the ultimate failure mode in all structures was local buckling of the compression flange and web. Local buckling was made more critical by stress concentrations, shear lag and bearing deformations caused by back-to-back bolted connections.

2. Selected literature review

Experimental research to determine the lateral-torsional buckling of cold-formed steel channel has been conducted by a number of researchers. These tests have been performed on single and pairs of plain and lipped channels with different cross section dimensions. Most of these tests used small cold-formed channels; typically the sizes that are normal used as purlins, and were restrained at various intervals within the length of the beam. The earliest tests to determine the lateral-torsional buckling strength of lipped channel beams were carried out by Winter et al. [4]. The purpose of these tests was to establish a brace spacing of channel beams that will achieve the same strengths as continuously braced channels. This study was the primary research work that led to the requirement of quarter-point bracing in the American Iron and Steel Institute Specification (AISI) for the Design of Cold-Formed Steel Structural members [5]. Quarter-point bracing system was specifically recommended for cold-formed steel channel and Z-flexural members to resist twisting and lateral buckling, when not attached to sheathing. In total Winter et al performed 18 tests on seven different cross-section dimensions, with web depths ranging from 102 to 203 mm, flange widths ranging from 63 to 102 mm, lip widths of 19 mm and the thicknesses ranging from 1.5 to 3.8 mm. Specific details of the dimensions of the sections were not given. For all tests, the span length of the beams was 3.5 m and the two concentrated loads were applied symmetrically about the mid-span, at a constant spacing of 0.66m.

Lateral braces were located at each end supports and two other lateral bracings were located symmetrically about the mid-span. The two intermediate bracings were varied by increasing the distance between them and keeping the span constant. In total, four different ratios of the distance between the braces to the distance between end supports or span length were tested, namely; 0m, 0,478m, 0.652m, and 1.0m. The ratios represented a fully braced beam, a single mid-span brace, a bracing at a quarter and three quarter location and a completely unbraced span, respectively. The bracing configurations were varied so that each system could be compared with a fully braced and an unbraced beam. The beams were subjected to eccentric loads, applied through the top flange at the flange-web junction. These tests showed a decrease in strength as the brace spacing was increased, implying that the critical strength is a function of the braced length. The provision of quarter point bracing appeared in all succeeding American Iron and Steel Institute Specification up to the Addendum [6], when it was replaced by a more exact procedure for calculating lateral torsional buckling of doubly, singly, or point symmetrical sections.

Hill [7] conducted an experimental and analytical investigation to determine the lateral-torsional buckling behaviour of 84.5x31.6x3mm equal-flanged cold-formed aluminium alloy channels. The aim of this study was to devise a rational procedure for designing such members.

Four channels of unbraced lengths of 508, 762, 1143, and 1651mm and corresponding flange-yield strength of 273.72, 309.58, 275.10 and 309.58MPa were tested.

A total of four strain-gauges were attached close to mid-span of the channel; two at the top flange and two at the bottom flange. For each flange, one strain-gauge was placed at the toe and the other one at the heel. The purpose of the strain-gauges was to quantify the variations in stresses in each flange so that it could be established whether lateral-torsional buckling occurred or not. The beams were subjected to two point loads to simulate a distributed load and tested in pairs so as to provide a stable test setup. Lateral restraints were applied at both end supports to restrict warping and at points of applied load. In all cases these lateral restraints were connected to the webs of the tested beams. An analysis of these stresses showed that no significant changes in stresses occurred in each flange, implying that there was no horizontal deflection. The beams were short enough to discourage lateral-torsional buckling. The beams with a longer unsupported length of 1651mm failed in the elastic range whilst the beams of shorter length failed by local buckling or crumpling of the compression flange. A comparison of the results for all tested beams shows that the moments and stresses decrease with increase in the unbraced length.

A total of 160 lipped and unlipped cold-formed steel beams were tested by Lindner and Kurth [8]. The purpose of the tests was to compare the strength of the beams, with the load applied at mid-span, either through the top web-flange junction or centroid of the top flange. In both cases the beam tests were simply supported and the testing programme used a single beam for each test. The results from these tests showed that the strengths achieved in beams with the load applied through the centroid were significantly lower than those achieved in beams with load applied through the web-flange junction. Bredenkamp et al. [9] conducted an investigation into the lateral buckling of cold-formed singly symmetric stainless steel beams. The purpose of this investigation was to compare the test moments with the moments predicted by the American Society of Civil Engineers' Specification for the Design of Cold-Formed Stainless Steel Structural Members [10]. Three different cross-sections of 64x33x10x1.6mm, 64x43x10x1.6mm and 64x53x10x1.6mm dimensions were tested. The lipped channel sections were chosen so that; (1) no local buckling occurs in the first set of beams, implying that the sections were fully effective, (2) no local buckling occurs in the compression flange of the lipped channel section of the second set of beams before the full section strength is reached and (3) local buckling occurs in the compression flange of the third set of beams. Two point loads were applied at both ends of the beam as cantilever so as to generate a uniform bending moment between the supports. The beams were simply supported to allow free rotation along the major axis; however they were restrained against lateral and warping at both support systems. The predominant failure mode observed in all three set of beam tests was lateral-torsional buckling. A comparison between the experimental moment and the theoretical moment, calculated based on tangent modulus method, showed that there was good agreement between the two moments.

Ellifritt et al. [11] conducted a study on the flexural capacity of discretely braced lipped channel sections, in order to understand the rationality of quarter-points bracing when the deck or sheathing is not attached to flexural members, as indicated in the American Iron and Steel Institute (AISI) Specifications [5-6]. The study was extended to establish whether this requirement was not more of a serviceability consideration than strength. A total number of 23 flexural tests, of 207x86x22x1.8mm, 209x87x21x2.4mm and 206x81x27x1.8mm channel sizes were conducted, and the corresponding yield strength for these channels were 438, 417 and 414MPa. All channel beam tests were 6m in length and were tested in pairs. The bracing conditions were varied to include continuous bracing, quarter-point, third-point, mid-point and no bracing along the span. A 25x25mm angle was used to brace the two beams. The difference between this set-up and Winter et al. [4] set-up is that in this case there was no bracing system at the end. This meant that the channels were free to rotate at the ends. In the 207x86x22x1.8mm, 209x87x21x2.4mm sections, the load points were not braced, however, bracing was provided at the load points in the 206x81x27x1.8mm sections.

All braced beams failed by distortional buckling of the compression flange. It was observed that six of the eight tests, which were braced at quarter and third-point, failed at less than the predicted lateral buckling load, while other bracing beams exceeded the predicted lateral buckling load. The work demonstrated that the quarter-point bracing, as required in the AISI Specification [6] was not needed, and that it may give the designer a false sense of confidence by predicting a large load that cannot be achieved. However, bracing was found to limit deflections and rotations in 206x81x27x1.8 beam tests (with one or both flange braced), when compared to completely unbraced beams. In contrast to the requirement proposed in the AISI Specification [6], the authors recommended a mid-span brace to control lateral deflections and rotations at service loads. This was an improvement to earlier work by Winter et al. [4] where quarter-point bracing was recommended.

Kavanagh and Ellifritt [12] investigated the design strengths of cold-formed channels in bending and torsion, when not attached to deck or sheeting. This work was an extension of the earlier tests on flexural capacity of discretely braced lipped channel sections by Ellifritt et al. [11]. In this study, ten tests were conducted with the load applied in the plane of the web, at the neutral axis. A 150x60x16x1.2mm cold-formed lipped steel channel section of 550MPa yield strength was used for all tests. Variables in tests included the span length, load position and brace location. All the beam tests were simply supported and tested in pairs. These two beams were connected to a rigid frame and oriented in opposite directions. No bracing was provided at the end support. The stresses and failure patterns of the members tested were found not to be simply dependent on the unbraced length alone, as has original been thought, but also on the number and location of the braces. Warping stresses created a stress gradient in the flange, and in the process inhibited flange buckling. This caused the section to be fully effective until failure is reached by one of the local failure modes. The tests results of highly braced channels or channels with several intermediate braces (2 or more intermediate braces within the span), in which torsional stresses are very high in relation to bending stresses, showed the AISI's [6] lateral-torsional buckling equations to be unconservative. Conversely, the lateral-torsional buckling equations underestimated the buckling strength of unbraced or midpoint braced members. The unbraced length of highly braced members (2 or more intermediate braces) did not fail due to lateral-torsional buckling as predicted by the specification, but by distortional buckling of the flange-lip intersection at a braced point.

Put et al. [13] performed lateral buckling tests on concentrically loaded (loaded through the shear centre), simply supported, unbraced single lipped cold-formed steel channel beams. The purpose was to develop a new approach for designing these members, since the previous methods were highly dependent on the buckling theory and tests of hot-rolled I-beams. A total of ten tests (five tests each) were conducted on two different cross-sections, 102x51x14.5x1.9mm and 102x51x12.5x1.0mm, of grade 450 and 550 steel, respectively. The selected span lengths for both sections ranged from 1700 to 2500mm. Simple supports prevented in-and out-of-plane deflections, but did not restrain in-and out-of-plane rotations, and warping displacements. The set-ups of the beam tests were grouped into two, according to the cross-section dimensions and the level of the applied load with respect to shear centre. The

102x51x14.5x1.9mm section was loaded at 40mm height below shear centre whilst the 102x51x12.5x1.0mm section was loaded at shear centre (0 mm height).

The test beams experienced large lateral deformations as the inelastic buckling load were approached, followed by catastrophic failures of the compressed element of the cross-section. In the longest 102x51x14.5x1.9mm channels the final mode of failure was a catastrophic local buckling failure at the compression flange-web junction, whilst in the shortest 102x51x12.5x1.0mm channels, the final failure mode was by catastrophic distortional buckling of the compression flange and lip. The experimental results showed that the strengths of the beams that failed in the negative direction (shear centre moved away from original centroid) by distortional buckling of the compression flange and lip were lower than the strengths of beams that failed in the positive direction (shear centre moved toward the original centroid) by local buckling of the flange-web junction. Failure in the positive direction increases the compression at the top flange-web junction, whilst failure in the negative direction increases the compression in the top flange lip. For all the tested beams, the moments at failure were lower when the beam lateral deflections increased the compression in the compression flange lip, and higher when they increased the compression in the flange-web junction. A comparison of the lateral buckling results with design codes AS 4100 for hot-rolled sections and AS/NZS 4600 for cold-formed sections showed that the predictions by AS 4100 were generally closer to the test results while the predictions by AS 4600 were too high for beams with thinner wall thickness, but too low for beams with thicker wall thickness. It was then suggested that AS 4100 be used to design cold-formed channels, instead of AS 4600.

Put et al. [14] conducted 34 tests on unbraced, simply supported cold-formed steel lipped channel beams of 2 different cross-sections, to address the problem of combined bending and torsion. To achieve this loading combination, the channels were eccentrically loaded at mid-span. The cross-sectional dimensions, test set-ups, end supports and loading positions were exactly the same as those in Put et al. [13]. In total the concentrated loads were applied at 8 different eccentricities. All beams failed either by local buckling of the compression lip or local buckling of the compression flange-web junction. Local buckling of the compression lip occurred when the eccentricity was negative, whereas a positive eccentricity always caused local buckling of the compression flange-web junction. This means that a negative eccentricity increased the compression in the top flange lip, resulting in the failure of the lip, whilst a positive eccentricity increased the compression in the top flange-web junction, resulting in the local failure of the flange-web junction. These tests showed that the beam strengths decrease as the load eccentricity increases and that the strength is higher when the load acts on the centroid side (positive) of the shear centre than when it acts on the negative side of the shear centre. Simple interaction equations were developed that can be used in the design of eccentrically loaded cold-formed channel beams.

The literature has helped to determine the unbraced length of the channels that can promote lateral-torsional buckling and the position where the load should be applied. Test results showed that the beam strengths decrease as the load eccentricity increases and that the strength is higher when the load acts on the centroid side (positive) of the shear centre than when it acts on the negative side of the shear centre. Cold-formed channels in bending and torsion demonstrated that their stresses and failure patterns are not only influenced by unbraced length, but also on the number and location of the braces. Unbraced lengths of highly braced members (2 or more intermediate braces) did not fail due to lateral-torsional buckling, but by distortional buckling of the flange-lip intersection at a braced point. The objectives of the tests are to examine the ability of the thin cold-formed angle cleat to restrain lateral-torsional buckling and to compare the test results with unfactored resistances from the North American Specification for the Design of Cold-Formed Steel Structural Members, AISI S100-13 [15]. In the first phase of this study, tensile coupon tests of the three cross-sections were conducted to obtain the material properties. The second phase involves experiments on the lateral torsional instability of single cold-formed channels.

3. Material properties

An anomaly in the South African steel construction industry is that all cold-formed lipped channels are made out of commercial quality steel. Although the chemical composition of this steel is controlled, no mechanical tests are performed on it. This implies that the channels are not graded. Commercial quality steel can be assumed to have a carbon content that will not exceed 0.3% and a carbon equivalent (CE) that will not exceed 0.51%. The carbon equivalent (CE) is expressed as:

$$CE = C + \frac{M_n}{6} + \left(\frac{C_r + M_o + V}{5} \right) + \left(\frac{N_i + C_u}{15} \right) \qquad (1)$$

where, C, M_n, C_r, M_o, V, N_i and C_u are the percentage of carbon, manganese, chromium, molybdenum, vanadium, nickel and copper in the steel, respectively. When Commercial quality steel has a CE value of less than 0.51%, then the steel is weldable. Since commercial steel is classified as "unidentified structural steel", according to SANS 10162-2 [16] and Clause 5.2.2 of SANS 10162-1 [17], its yield stress and tensile strength are taken as not more than 200 and 365MPa, respectively. Currently the sections that are available in the industry have a depth that varies from 75mm to 300mm, a flange width that varies from 50mm to 100mm, a lip of 20mm and a thickness of the sections that varies from 2.0mm to 4.5mm.

A total of 18 coupon test specimens of 300x75x20x3mm, 300x75x20x2.5mm, and 300x75x20x2.0mm channel sections cross-sections were conducted to obtain the material properties. The coupon test specimens were cut from the web and flange of the channel sections used, and prepared and tested in a 100kN capacity displacement controlled testing Instron, according to the guidelines provided by the British Standard, BS EN ISO 6892-1 [18]. Web coupon tests are expected to represent the material properties of the steel sheet from which the sections were rolled. In order to calculate the area, and subsequently the stresses, the thickness and width of the reduced section of the coupons were measured and recorded on the computer system. A 50mm gauge length was marked onto the tensile test specimens before testing in order to measure the axial elongation after fracture. The longitudinal strain gauges, attached to the coupon at the centre of each face, were used to determine the strains. Initially, a small tensile load was applied to the coupon until it was properly gripped. Thereafter, the required load was applied at a constant rate of 3.0mm/min until failure.

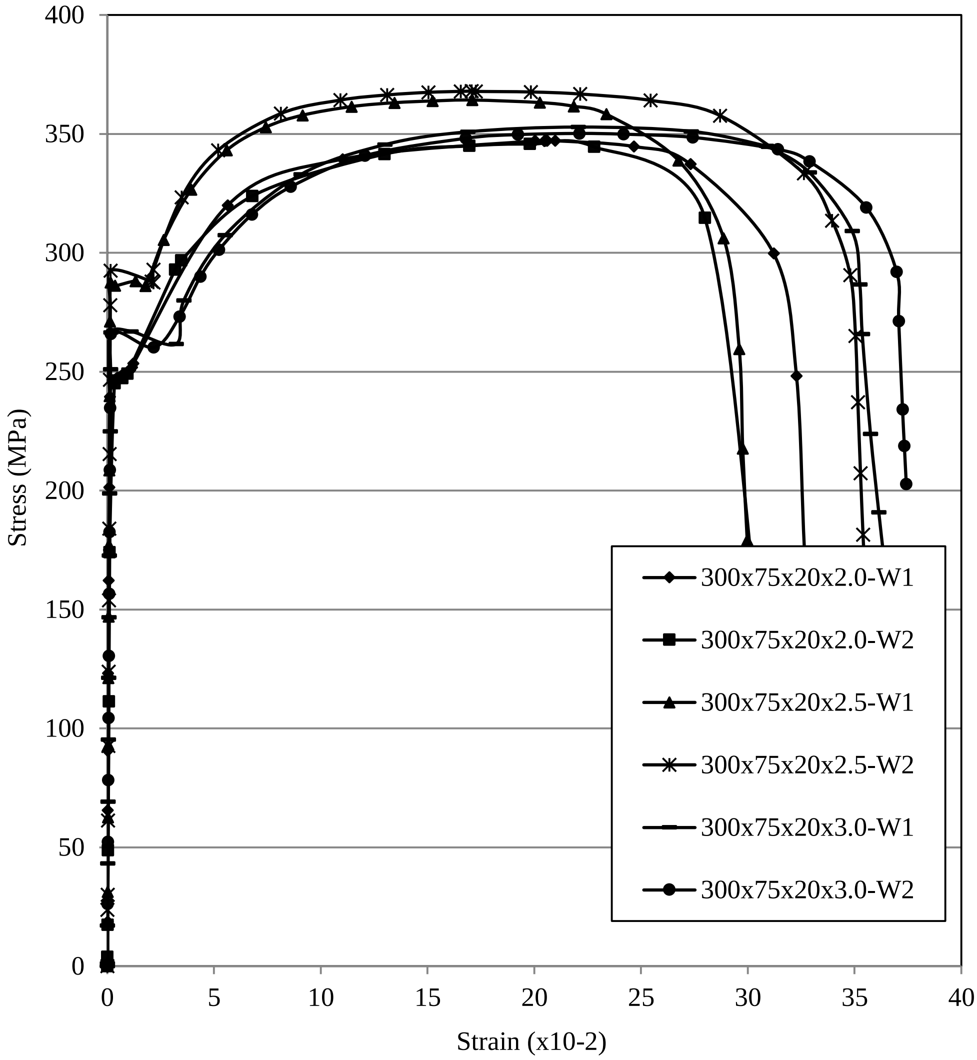


Figure 3 Typical stress-strain graphs

Table 1. Average material properties

Frames	Channel sections	f_y (MPa)	f_u (MPa)	f_u/f_y	ε_f (%)	E (GPa)
1	300x75x20x2.0	246.60	346.05	1.40	28.35	206.35
2-4	300x75x20x2.5	287.30	364.05	1.27	28.80	201.61
5-7	300x75x20x3.0	265.70	351.55	1.32	30.29	205.58

Figure 3 shows typical stress-strain relationships of the coupons. These were derived from the load-elongation relationship, using its original cross-sectional area and the gauge length. In this figure, W1 represents Web Coupon 1 and W2 represents Web Coupon 2. The yield stress, ultimate stress and modulus of elasticity of the steel were determined from these stress-strain curves. The average yield strength, f_y, average ultimate strength, f_u, and the elastic modulus, E, are summarized in Table 1. In this table, ε_f is the ductility of the steel. Since 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, as recommended by SANS-10162-2 [16], this means that the material properties of the channels achieved the expected ductility. The ratio f_u/f_y determines the ductility of the steel. A high ratio of f_u/f_y implies that the steel section is very ductile. The elastic modulus and yield strength of the cold-formed steel are used to calculate the code-predicted lateral-torsional buckling moment resistance. In order to be consistent, only coupon results from the web were used in these calculations.

4. Experimental programme of the test specimens

4.1 Preparation

The size of the lipped cold-formed channels used in this investigation are 300x75x20x2.0mm, 300x75x20x2.5mm and 300x75x20x3.0mm, and the purlin and angle cleat section are 100x50x20x2.0mm and 100x75x20x3.0mm, respectively. The three sections are among the largest group of lipped cold-formed sections, manufactured in the South African steel industry. The choice for testing these large sections was determined by the desire to load the angle cleats to their limit. The cross-section dimensions of the selected sections were measured at mid-span and at both ends of the beam using a pair of vernier callipers to determine the nominal dimensions. These measurements were found to be very close to the nominal ones provided by the supplier. The beam span varied from 4.80m to 6.4m. The length of all beams was made 200mm longer than the beam span to allow for the lateral restraints at each support. In both frames, the selection of the beam spans was made so that lateral-torsional buckling would occur, and also, to produce a range of beam slenderness ratios. Based on the current laboratory setting, the longest span length was limited to 6.4m.

4.2 Test set-up

Several pairs of channel beams and angle cleats were prepared in order to examine the ability of thin cold-formed steel angle-cleats to restrain the lateral-torsional buckling of the channel beams. The beams were tested under two point loading, as illustrated by the schematic diagram in Figure 4, in order to provide a constant moment between the applied loads. This loading arrangement enables that pure bending failure only is experienced in the middle unbraced length. The supports were designed to ensure that the beam test is simply supported. Simple supports prevent in-and out-of-plane deflections, but do not restrain in-and out-of-plane rotations, and warping displacements. Angle cleats provided lateral and torsional restraints at the loading points. Details of the span and points of bracing are given in Table 2.

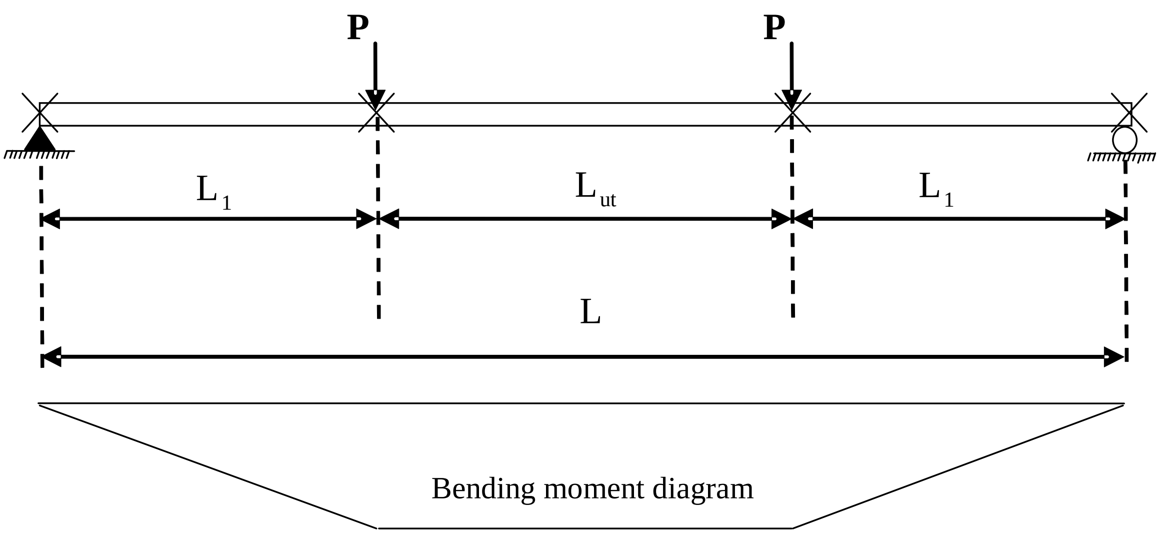


Figure 4. Restraints and support system

The beams sections and the middle unbraced lengths were varied in order to vary the slenderness ratios. The middle unbraced lengths were selected based on the recommendation of Section C3.1.2.1 of the North American Specification for the Design of Cold-Formed Steel Structural Members, AISI S100-13 [15], on maximum unbraced length, to promote lateral-torsional buckling. In this standard, the effective proportional limit or the upper limit of elastic buckling is assumed to be equal to one-half the maximum stress. This means that lateral-torsional buckling is considered to be elastic up to a stress equal to $0.56f_y$. The inelastic region is defined by a parabola, from $0.56f_y$ to $1.11f_y$. The upper limit stress of $1.11f_y$ is based on the partial plastification of the section in bending [19]. A flat plateau is created by limiting the maximum stress to f_y, which enables the calculation of the maximum unsupported length for which there is no stress reduction due to lateral-torsional instability. For lateral-torsional buckling to occur, the internal length should be greater than the maximum laterally unbraced length (L_{uc}). For a channel section, the maximum unbraced length (L_{uc}) in Equation 2 is determined by setting the elastic critical lateral-torsional buckling stress, $f_e = 2.78f_y$ and unbraced length $L_{uc} = L_y = L_t$. Parameters L_y and L_t are unbraced lengths for bending about the y-axis and z-axis, respectively.

$$L_{uc} = \left\{ \frac{GJ}{2C_1} + \left[\frac{C_2}{C_1} + \left(\frac{GJ}{2C_1} \right)^2 \right]^{0.5} \right\}^{0.5} \tag{2}$$

where,

$$C_1 = \frac{7.72}{AE} \left(\frac{K_y f_y S_{xc}}{C_b \pi r_y} \right)^2 \qquad \text{and} \qquad C_2 = \frac{\pi^2 E C_w}{(K_t)^2}$$

In Equation (2), G is the shear modulus, J is the St. Venant torsional constant, K_y is the effective length factor for bending about the minor axis, K_t is the effective length factor for torsional buckling, S_{xc} is the effective section modulus about the x-axis, A is the area, r_y is the radius of gyration about the minor axis, and C_w is the warping constant and the bending coefficient, $C_b = 12.5M_{\max} / (2.5M_{\max} + 3M_A + 4M_B + 3M_C)$, where, M_{max} is the absolute value of maximum moment in unbraced segment, M_A is the absolute value of moment at quarter point of unbraced segment, M_B is the absolute value of moment at centreline of unbraced segment and M_C is the absolute value of moment at three-quarter point of unbraced segment. The tested internal length varied from 1.8m to 2.4m. Table 2 gives the span (L), calculated maximum unbraced length (L_{uc}), length of the tested internal unbraced length (L_{ut}), length of the external unbraced length (L_1) and the slenderness ratio $(\lambda = KL_{ut}/r_y)$ of the internal unbraced length of the tested beams.

Table 2. Unbraced Length of tested beams.

Frames	Channel sections	L (mm)	L_{uc} (mm)	L_{ut} (mm)	L_1 (mm)	r_y (mm)	KL_{ut}/r_y
1	300x75x20x2.0	6000	1775.72	2200	1900	25.7	85.60
2	300x75x20x2.5	5800	1621.53	1800	2000	25.4	70.87
3	300x75x20x2.5	6000	1621.53	2000	2000	25.4	78.74
4	300x75x20x2.5	6400	1621.53	2400	2000	25.4	94.49
5	300x75x20x3.0	4800	1694.56	1800	1500	25.1	71.71
6	300x75x20x3.0	6000	1694.56	2200	1900	25.1	87.65
7	300x75x20x3.0	6000	1694.56	2400	1800	25.1	95.62

A schematic arrangement of a pair of the channel beams, angle cleat and purlin is shown in Figure 5. Based on previous research, a pair of channels was used in the test set-up, to allow for interaction between the beams, and provide stability to the test assembly. This idea was selected based on previous research [1, 2, 3]. The channels are oriented in the same direction as this offers greater stiffness than having the channels oriented in different directions [1, 2, 3]. The size of the purlin of 100x50x20x2mm and spacing of the channel beams of 1.84m were selected so that they are suitable for the short spacing between the beam channels. Two purlins were used in all the tests to connect the two beams. As indicated before a 100x75x20x3mm angle cleat was used to restrain lateral-torsional buckling of the beams. The bolts used to attach the angle cleat to the purlin were aligned diagonally in order to generate enough rotational moment of resistance.

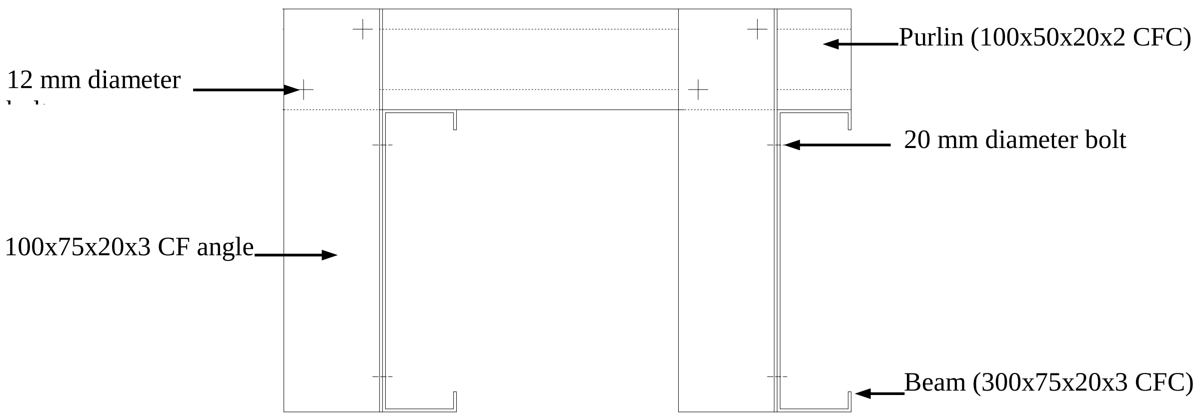
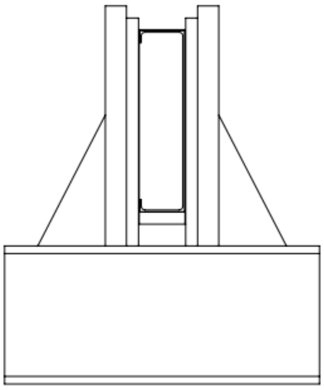


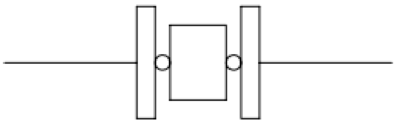
Figure 5 Schematic cross-section arrangement of the frame

A photograph of the test set-up is shown in Figure 6. The test rig included specially designed spreader beams and a loading system to facilitate the testing of beams under two-point loading. The simply supporting system allowed in-and out-of-plane rotations, and warping displacements, but prevented in-and out-of-plane deflections, as illustrated in Figure 7. To create the spacing of the channel beams, the loading system was offset from the original centre of the Instron. The channels were stiffened by a plate and an angle at the loading point to avoid local failure. The angle was bolted on the web, inside the channel, at each loading point, with the same M20 bolts that were used to connect the angle-cleat. This prevented the top flange of the channels from failing prematurely.

Figure 6 Test set-up



(a) Longitudinal view



(b) Plan view

Figure 7 Details of the supports

4.3 Instrumentation and test procedure

Several measurements were recorded during the beams tests, and these include in-plane and out-of-plane deflection, strains and rotation. Both in-plane and out-of-plane deflection were measured using Linear Variable Differential Transducers (LVDTs). As illustrated in Figure 8(a), in-plane was measured using one LVDT and out-of-plane deflection was measured using two LVDTs. In order to determine the moment-curvature behaviour of the beams, bending strains were recorded by means of strain gauges. For each channel, one strain gauge was placed at the centre of the top compression flange, and another one placed at the centre of the bottom tension flange. Rotation was monitored using a rotation gauge (also known as an AccuStar Electronic Clinometer), as shown in Figure 8(b). To enable the multimeter to read zero in the unloaded condition, the two holes of the rotation gauge were aligned vertically. All measurements were recorded at the mid-span of the beams by means of a data-logging system.

(a) LVDTs

(b) Rotation gauge

Figure 8 Instrumentation

As soon as all the instruments were set and calibrated, two equal loads were applied simultaneously to the frames, using a 250kN hydraulic Instron. The loads were applied at the centre of the top flange. This loading arrangement produces a lower moment capacity than if the load had been applied at the shear centre. Both beams were loaded at the rate of 0.5mm/min, so that the behaviour and failure patterns of the beam could be well observed during the test. The test was stopped once the channels had failed.

5. Failure modes

The final mode of failure in all the beams tested was a catastrophic distortional buckling of the web and flange. This failure mode occurred after extensive lateral-torsional buckling of the middle unbraced segment. Lateral-torsional buckling was largely influenced by the length of the internal segment, the thickness of the channels and the loading position. As illustrated in Figure 9, the channel buckled out-of-plane and twisted about the shear centre so as to relieve the compression on the stiffening lip. Lateral-torsional buckling was critical in these sections because the channels exhibited low torsional rigidity. As evidence that the angle cleats were able to restrain the channels, the outside segments and the restraining angle cleats remained vertical.

(a) Top view (b) Side view

Figure 9 Lateral-torsional buckling of the channels

Distortional buckling is characterized by the rotation of the top, compressed lip-stiffened flange about the flange-web junction, in cross-sections with edge stiffened elements, such as lipped cold-formed channel beams sections. This usually occurs if the lip does not have enough stiffness to prevent the flange from rotating. The rotation can cause the flange to either move outward or inward depending on the nature of the load, supporting system or imperfections. At ultimate failure, both rotations can be accompanied by the bending of the web. The wave length of this mode of failure is between that of local and overall member buckling, which makes it a practical beam length. In the thicker channels (300x75x20x3.0mm and 300x75x20x2.5mm), distortional buckling caused the web to deflect inward towards the centroid and the top compression flange to deflect upwards (Figure 10). Conversely, distortional buckling caused the web of the 300x75x20x2.0mm channel beams to deflect outward towards the shear centre and the top compression flange to deflect inward towards the neutral axis (Figure 11). This behaviour was probably influenced by the small thickness of this section. Similar failure mode was observed by Ellifritt et al. [11] and Kavanagh and Ellifritt [12] on flexural capacity of discretely braced channel section in bending. In all beams distortional buckling of the compressed flange occurred after significant lateral-torsional deformations.

(a) Inward deflection of web

(b) Outward deflection of flange

Figure 10 Outward distortional buckling

Figure 11 Inward distortional buckling

6. Test results

The experimental and code-predicted results of the single channels are given in Table 3. In this table P is the maximum applied load (include the weight of the spreader), M_{t1} is the maximum moment applied to the tested specimens, M_y is the yield moment, M_d is the distortional buckling moment of resistance and M_r is the lateral-torsional buckling moment resistance, determined using the North American Specification for the Design of Cold-Formed Steel Structural Members, AISI S100-13 [15]. Both the yield moment and the lateral-torsional 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 lateral-torsional buckling moment of resistance is established from the elastic critical lateral-torsional buckling stress, which is then transformed to a critical buckling stress, taking into account the inelastic strength of the channels. Resistance to distortional buckling is evaluated from the elastic distortional buckling stress, and the corresponding distortional buckling moment is calculated using the gross section modulus of the cross-section, accounting for the inelastic and post-buckling strength of the channels. An effective length factor of 1 is assumed for bending about the minor axis and a moment-gradient factor of 1 is used because of the uniform bending moment diagram. The ultimate moments of the beams are compared with the predictions from the specified code.

Table 3 Test results

Frames	Channel sections	L_u (mm)	f_y (MPa)	P (kN)	M_t (kNm)	M_y (kNm)	M_d (kNm)	M_r (kNm)	$\frac{M_{t1}}{M_y}$	$\frac{M_{t1}}{M_d}$	$\frac{M_{t1}}{M_r}$
1	300x75x20x2.0	2200	246.6	11.04	18.03	17.72	15.47	16.9	1.02	1.17	1.07
2	300x75x20x2.5	1800	287.3	17.33	29.86	26.18	23.25	25.64	1.14	1.28	1.16
3	300x75x20x2.5	2000	287.3	13.59	21.03	26.18	23.25	24.96	0.80	0.90	0.84
4	300x75x20x2.5	2400	287.3	11.00	15.66	26.18	23.25	23.48	0.60	0.67	0.67
5	300x75x20x3.0	1800	265.7	26.34	38.4	31.51	29.92	30.25	1.22	1.28	1.27
6	300x75x20x3.0	2200	265.7	17.04*	28.56	31.51	29.92	28.69	0.91	0.95	1.00
7	300x75x20x3.0	2400	265.7	17.36	30.07	31.51	29.92	27.69	0.95	1.01	1.09

*anomaly

As anticipated, the moment capacities (M_{t1}) of all beams in Table 3 decrease with increase of the middle segment length. The table also shows that the yield moment (M_y) is not significantly larger than the buckling moment of resistance (M_r). This is clearly supported by the fact that the internal unbraced length (L_{uc}) of all beams was marginally greater than the yield unbraced length (Table 2). In the frames with 300x75x20x3.0mm sections, the tests moment varied from 30.07kNm for the 2.4m middle segment to 38.40kNm for the middle segment of 1.8m. For the frames with 300x75x20x2.5mm sections, the moment capacities varied from 15.66kNm for the middle segment of 2.4m to 29.86kNm for the 1.8m middle segment. Table 3 shows that the channels of Frames 1, 2 and 5 achieved the yield moment. Evidence that the channels of these frames actually yielded is illustrated in the graphs in Figures 11, 12 and 13. As reflected in Table 2, yielding was promoted by the shorter lengths of the channels. A slight increase in length of these channels resulted in a substantial decrease in capacity (Frame 3, 4, and 7).

As discussed in Section 5, the final failure mode was distortional buckling. Distortional buckling manifests itself as the rotational deformation of the flange and lip of the channels about the web-to-flange junction. A comparison of the test moments versus the code-predicted moments shows that in all the frames tested the ratio of the test moment versus the distortional buckling moment of resistance is larger than the ratio of the tests moments versus the yield moment or the lateral-torsional buckling moment of resistance. This supports the view that distortional buckling is in fact the critical failure mode, and it occurred at moments less than the predicted lateral-torsional buckling moment of resistance. Distortional buckling is more critical in frames with shorter unbraced lengths and thicker channels. As presented in Table 3, Frame 5 with 300x75x20x3.0 channels and 1800mm unbraced length experienced much more distortional buckling than Frame 4 with 300x75x20x2.5 channels and 2400mm unbraced length. This work partly agrees with the findings of Ellifritt et al [11], and Kavanagh and Ellifritt [12]. Both have also shown that beams that are discretely braced may fail either by lateral-torsional buckling between braces or by distortional buckling. However, in these tests distortional buckling occurred at mid-length after extensive lateral-torsional buckling of the internal unbraced length, and not at or near the braced point. Distortional buckling was probably the governing mode of failure in these beams because the lips were not sufficiently stiff enough to stabilise the flanges, the unbraced lengths were not long enough to promote lateral-torsional buckling only, and no rotational restraint was provided to the compression flange.

For all frames the ratio of the test-to-predicted moments tends to be larger for sections of shorter length, and decrease as the length of the channels increases. It is very important to note that in all the results achieved in Table 3 the angle cleat did not fail. The angle cleat configuration remained vertical when the internal segment of the beam buckled. Until distortional buckling took place, lateral-torsional buckling did not contribute to a significant loss of moment of resistance.

7. Moment-deflection, curvature and rotation graphs

The moment-deflection, curvature and rotation graphs for the channels are given in Figures 12, 13 and 14. The maximum moment in these graphs do not include the spreader load. Initially, all channels in these figures show linear-elastic response, but became progressively inelastic as the applied load approached the maximum capacity of the channels. Inelasticity was more visible in the channels with shorter internal unbraced length and larger thickness, such as the 1.8m length of the 300x75x20x3.0mm and 300x75x20x2.5mm sections. As indicated above, all the test beams experienced a catastrophic distortional buckling. Catastrophic distortional buckling shook the entire structure vigorously as the channels were swiftly lifted and twisted. This sudden motion was accompanied by a loud noise. Evidence of catastrophic distortional buckling is shown by a sudden vertical kink in the moment-deflection, curvature and rotation graphs in Figures 12, 13 and 14. Immediately after this sudden kink, the channels quickly shed-off of the load. The maximum moments discussed in this paper excludes the vertical kink.

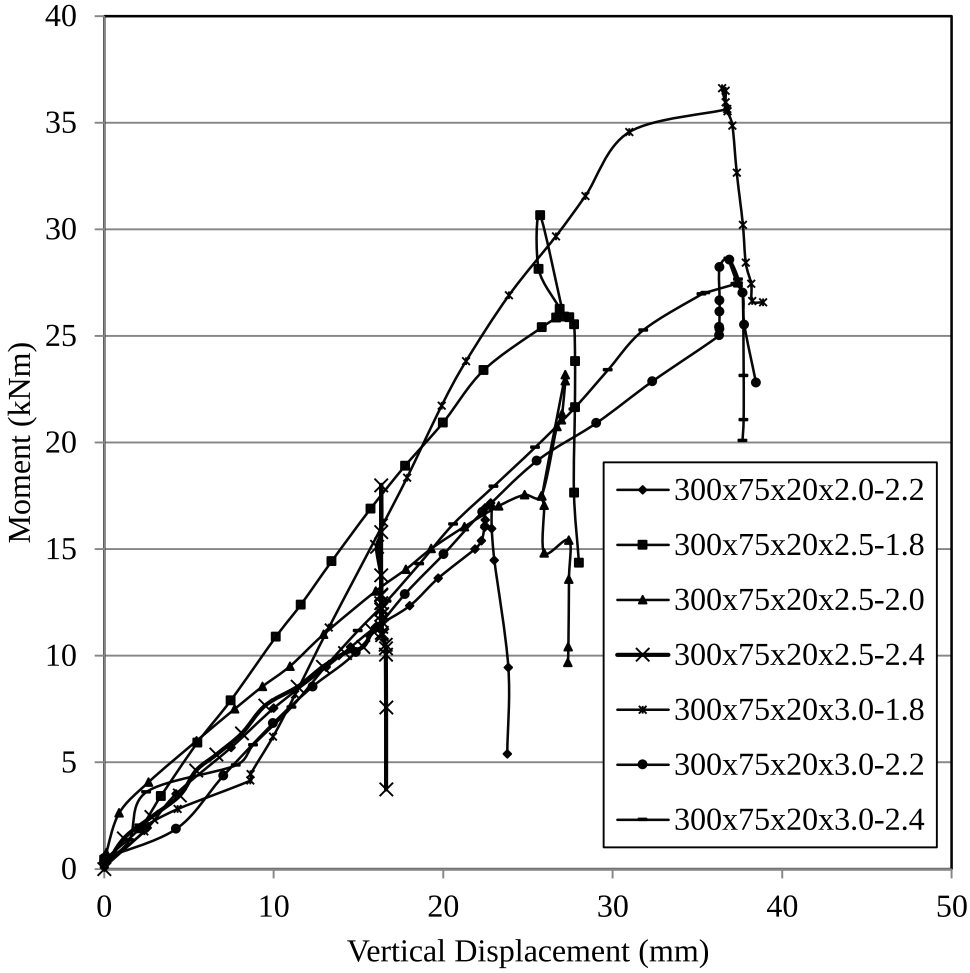


Figure 12 Moment-Vertical Deflections

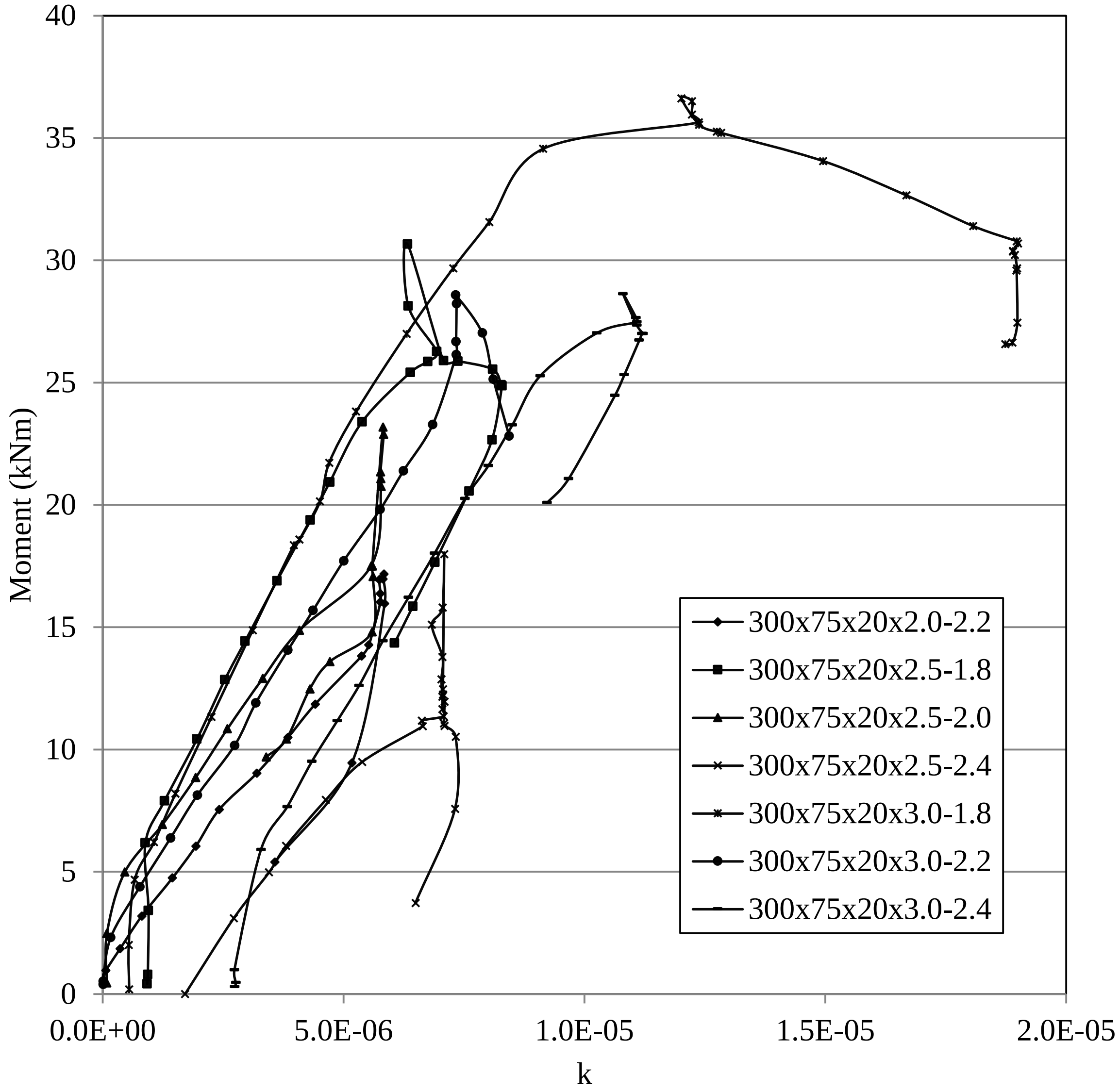


Figure 13 Moment versus curvature graphs

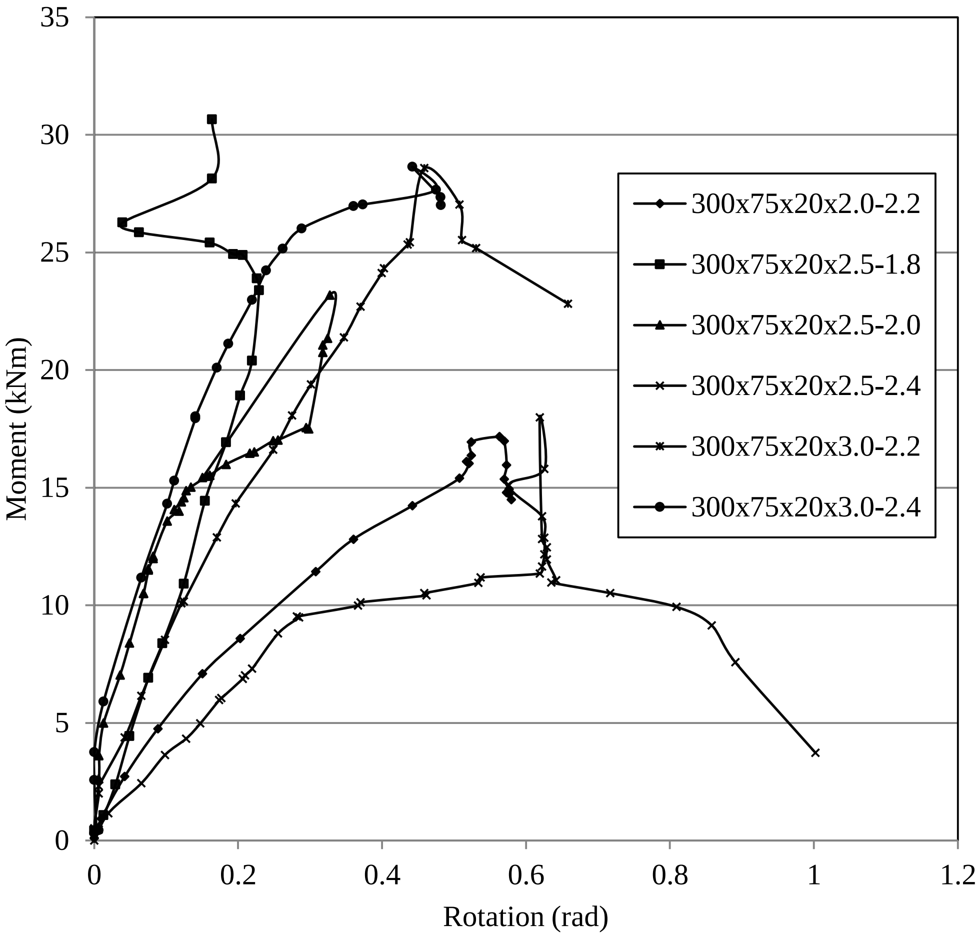


Figure 14 Moment-rotation

The magnitude of the kink, caused by distortional buckling is given in Table 4. In this table M_{t2} is the maximum moment after distortional buckling. For the same size of channel section and yield strength, the magnitude of the kink increases with increase in length. In addition, the magnitude of the kink tends to be lower for channel sections with thicker sections and lower yield strength.

Table 4. Magnitude of the kink.

Frames	Channel sections	L_u (mm)	f_y (MPa)	P (kN)	M_{t1} (kNm)	M_{t2} (kNm)	M_{t2}- M_{t1} (kNm)
1	300x75x20x2.0	2200	246.6	11.04	18.03	20.98	2.95
2	300x75x20x2.5	1800	287.3	17.33	29.86	34.66	4.80
3	300x75x20x2.5	2000	287.3	13.59	21.03	27.18	6.15
4	300x75x20x2.5	2400	287.3	11.00	15.66	22.00	6.34
5	300x75x20x3.0	1800	265.7	26.34	38.4	39.51	1.11
6	300x75x20x3.0	2200	265.7	17.04*	28.56	32.38	3.82
7	300x75x20x3.0	2400	265.7	17.36	30.07	31.25	1.18

*anomaly

Figure 14 and Table 5 show that there was significant torsional buckling in the unbraced length of the beams tested. It is clearly observed from Table 5 that the rotation was influenced by the unbraced length and thickness of the channels. In the beams of the same thickness, the rotation at maximum load was smaller for a shorter unbraced length than a longer unbraced length. For example, beams with an unbraced length of 1.8m, 2.0m and 2.4m and a thickness of 2.5mm achieved rotations of 0.23 radians, 0.33 radians, and 0.62 radians, respectively. Similarly, beams with an unbraced length of 2.2m and 2.4m and a thickness of 3.0mm attained rotations of 0.46 radians and 0.47 radians, respectively. As for the thickness, the 2.2m unbraced length-beam of 2mm thickness rotated more than the 2.2m unbraced length-beam of 3mm thickness, due to smaller thickness. In the same way, the 2.4m unbraced length-beam of 2.5mm thickness rotated more than the 2.4m unbraced length-beam of 3mm thickness. The rotation values clearly show the ability of the beams with high thickness to tolerate larger torsion before reaching the buckling moment.

Table 5. Rotation

Frames	Specimen	L_{ut} (mm)	(t mm)	α (rad)
1	300x75x20x2.0	2200	2.0	0.56
2	300x75x20x2.5	1800	2.5	0.23
3	300x75x20x2.5	2000	2.5	0.33
4	300x75x20x2.5	2400	2.5	0.62
5	300x75x20x3.0	1800	3.0	-
6	300x75x20x3.0	2200	3.0	0.46
7	300x75x20x3.0	2400	3.0	0.47

8. Conclusion

This paper has presented the results of an experimental investigation into the flexural strength of 7 frames of single lipped cold-formed steel channel sections, subjected to two- point loading and restrained by angle cleats. From the tests the following conclusions are made:

- The final failure mode of all the beams was by catastrophic distortional buckling of the web and flange. This mode of failure occurred after extensive lateral-torsional buckling behaviour. Distortional buckling caused the top half of the web and the compression flange to rotate outward in the thicker channels (300x75x20x3.0mm and 300x75x20x2.5mm) and inward in the 300x75x20x2.0mm channels.

- The modes of failure experienced by the channels did not affect the restraining angle cleat. The angle cleat configuration remained vertical as the unbraced length was buckling. The capacity reached by the channels shows that an angle-cleat can be used as an effective restraining system to resist lateral-torsional buckling. This means that such a system can be used without the need of fly-bracings, as is normally done in practice to restrain torsional instability.

- The use of a stiffening element at the point of the applied load prevented local failure. This allowed the test strength of the channels to increase significantly. The two 12 mm diagonally aligned bolts, connecting the purlin and the angle cleat were able to generate enough rotational moment of resistance.

- In all the frames tested the ratio of the test moment versus the distortional buckling moment of resistance is larger than the ratio of the tests moments versus the yield moment or the lateral-torsional buckling moment of resistance. This supports the view that distortional buckling is in fact the critical failure mode, and it occurred at moments less than the predicted lateral-torsional buckling moment of resistance. Distortional buckling is more critical in frames with shorter unbraced lengths and thicker channels.

Since this study was experimental, further work on these sections will focus on a numerical study. Of important significance in this study will be the interaction of lateral-torsional buckling and distortional buckling.

8. Aacknowledgement

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