

Experimental, theoretical and numerical study on thin walled steel single and compound channel sections in axial compression

G Beulah Gnana Ananthi^{a*}, S Vishuvardhan^b & G M Samuel Knight^a

^aDivision of Structural Engineering, CEG Campus, Anna University, Chennai 600 025, India

^bCSIR-Structural Engineering Research Centre, CSIR Campus, Taramani, Chennai 600 113, India

Received 6 February 2014; accepted 18 June 2015

This investigation presents a detailed description of experimental, theoretical and numerical studies of the flexural buckling strength of cold-formed steel (CFS) channel stub columns. A total of 24 single channels, double channels welded back to back and double channels welded as box sections both with and without lip stub columns are tested till failure with hinged ends and their behavior is studied. An attempt is made to study the effect of lip, and the effect of symmetricity of cross-section on the ultimate load carrying capacity. The flat width to thickness ratio of all the specimens are kept less than the limiting value as per IS 801-1975. Geometric and material non-linearity have been included in the finite element model. The design column strengths calculated using the finite element analysis (FEA) are compared with the IS: 801(1975), BS: 5950 (Part 5) and NAS manual. It is shown that the design column strengths calculated based on the codal specifications generally are conservative with experimental and numerical strengths.

Keywords: Cold-formed, Lipped channels, Double channels, Box sections, Load carrying capacity, Finite element analysis

Cold-formed sections have widespread use in the building industries. They are used where the loads are not large and thus where light members will suffice, achieving substantial economics. In addition, they are easily fabricated into a great variety of shapes, and may be formed in sheet metal shops which are widely available. Cold-forming are made from flat sheet of strip, either by brake forming or by roll forming. In contrast to hot-rolled members who are subjected to residual cooling stresses, cold-formed sections are subjected to strain hardening caused by cold-forming, which may significantly affect the structural performance. Even more important, while the width-thickness ratios of the component plate elements are limited in hot-rolled sections by the manufacturing processes, the cold-forming process allows for practical unlimited width-thickness ratios. With large width-thickness ratios, local plate buckling can occur prior to attainment of the maximum member strength. Anil Kumar and Kalyanaraman¹ has studied the use of direct strength method (DMS) for CFS compression members without stiffeners and a comparative study with test results and effective width method (EWM) has shown that DSM estimates the strength within an acceptable accuracy. Numerical and theoretical

studies on CFS unlipped channels which are subjected to axial compression were done by Beulah *et al.*² and it is observed that the design strength predicted by the NAS is conservative for axially loaded columns. Zhang and Young³ have shown that the design strengths predicted by the modified direct strength method are generally in good agreement with the ultimate loads of the built-up open section columns. Stone and Laboube⁴ have studied the behaviour of CFS built-up –I sections and has established that the provisions of the North American specification are conservative for thick members. Compression test on CFS built-up back to back channels stub columns was done by Ting and Lau⁵ and they have concluded that the EWM from the North American specifications gives a better prediction than DSM. An attempt was made by Vishuvardhan and Knight⁶ to study the effect of lip, thickness of the section, size of the section, material yield strength, and section symmetricity on the ultimate load carrying capacity and the mode of failure. Studies executed on the axially loaded cold-formed built-up C-Channels by Whittle and Ramseyer⁷ show that use of the modified slenderness ratio is more conservative for longer and thickened built-up members. A column buckling curves, based on experiments on short, intermediate and slender compression members has been provided by Young⁸

*Corresponding author (E-mail: beulah28@annauniv.edu)

from the work done on CFS built-up closed sections with intermediate stiffeners. From the literature, it is concluded that that limited work has been reported on cold-formed steel channel stub columns and the results of experimental investigation on double channels are hardly available. The present study was conducted to analyze the behaviour of axially loaded single and double channels. The present investigation has been under taken to study the behavior of cold formed channel stub columns, which separately investigates: (i) The effect of lip on the ultimate load carrying capacity; (ii) The effect of lip on the ultimate load carrying capacity; (iii) The effect of symmetry of cross section on the ultimate load carrying capacity; (iv) The behavior of open web and closed web sections in the elastic as well as in the plastic ranges to loading and (v) The comparison of observed ultimate load carrying capacity of the specimens with the predicted values according to the three international codes.

The first objective of this investigation is to experimentally study the behavior of cold-formed steel channel stub columns in axial compressive force in the elastic as well as in the plastic ranges of loading. Two different thicknesses of channels both with and without lip were tested up to failure. The loads were applied through the centre of gravity of the cross-section and the behavior was studied. Secondly, a finite element analysis was conducted on a FE model to investigate the strength and behavior of cold-formed single and double channels and they were analyzed taking into account of both material and geometrical nonlinearity. Finite element analysis of cold-formed steel structures plays an increasingly important role in engineering practice, as it is relatively inexpensive and time efficient compared with physical tests, especially when a parametric study of cross-section geometries is involved. The third objective of this paper was to compare the column strengths obtained from the finite element analysis with the design column strengths calculated using various international codes like BS: 5950-Part5³, IS: 801-1975⁶ and NAS Manual⁸ for cold-formed steel structures.

Experimental Procedure

Test specimens

The specimens were tested as single channels, channels welded back to back and channels connected as box sections to study their behavior as shown in Fig. 1.

Material properties

Totally, four tension tests were conducted according to IS: 1608 Part-1 (2005)⁹ to determine the yield stress, ultimate stress and percentage of elongation. The specimen used in the present investigation were fabricated from cold-formed steel sheet of thickness 2.1 mm and 3.3 mm. Since cold-formed steel yields gradually, offset method was used to determine the yield stress. The common values of Young's modulus, yield stress, ultimate stress and percentage elongation obtained from 0.2% offset are shown in Table 1.

NAS¹⁰ recommends the yield point of the steels to range from 172 to 482 N/mm². Similarly NAS¹⁰ standards specifies the ultimate tensile strength of the steel to range from 289 to 584 N/mm² and the ratios of tensile strength to yield strength ranges from 1.21 to 1.8. The recommended NAS¹⁰ minimum percentage elongation ranges from 12 to 27. Figure 2 shows the typical stress-strain behaviour for the tension coupons. It is observed that the yield strength, ultimate tensile strength and percentage elongation of the chosen steel falls within the ranges specified by the NAS manual¹⁰ except for the percentage elongation which is varying by 1%.

It was intended to have sections that were fully effective, and all column failures are due to flexural buckling around the weak axis of the sections. According to IS 801-1975¹¹, the limiting flat width to thickness ratio for load determination for open section is $1435/\sqrt{f}$ and for closed section is $1540/\sqrt{f}$. Based on this criterion, all the specimens used in this investigation were proportioned so that the flat width to thickness ratio of the component plate elements of the section was smaller than the limiting value. It was also assumed that the minimum depths of lips provided are according to IS 801-1975¹¹. The length of the stub columns were

Table 1—Material property of coupons

Steel	Thickness (mm)	E (N/mm ²)	σ_y (N/mm ²)	σ_u (N/mm ²)	σ_u/σ_y	Percentage elongation
Type 1	2.1	211000	415	495	1.91	10
Type 2	3.3	201000	250	350	1.40	11

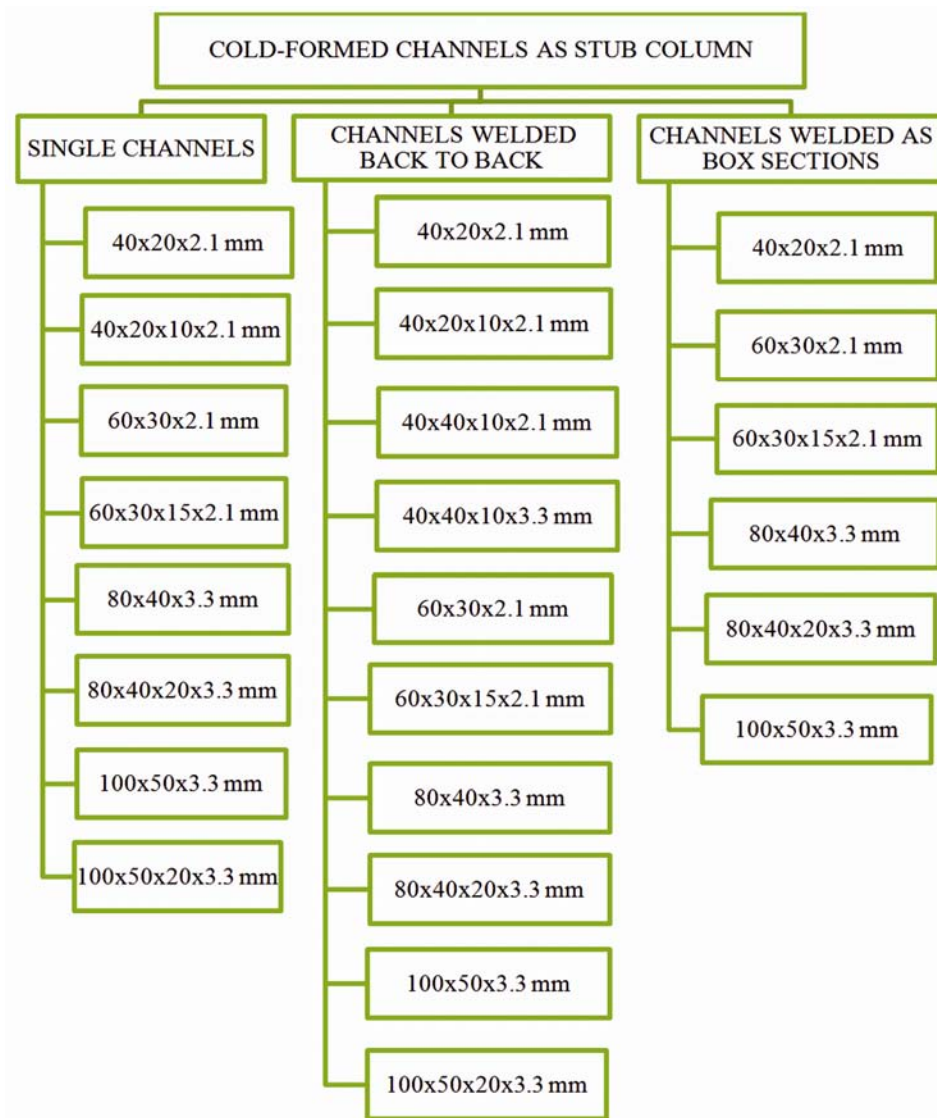


Fig. 1—Details of study conducted

determined based on the recommendations of the technical memorandum given in Structural Stability Research Council guide. In the memorandum it was suggested that, the length of the stub column should not be less than three times the largest dimensions of the sections, and should not be greater than twenty times the radius of gyration about the weak axis to the section. The object of choosing this length is to ensure that the stub column is short enough so that the influence of overall buckling is minimized, but long enough so that the ends effects can be neglected.

End connection

Totally twenty four stub columns of sizes and shapes are cut to the required length and to 6 mm

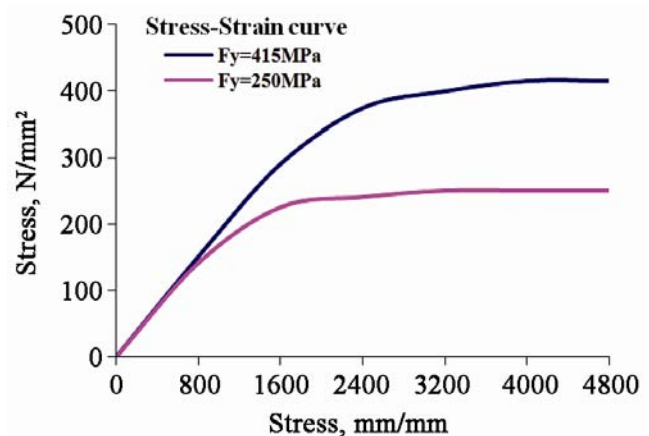


Fig. 2—Stress-strain behavior of tensile coupons

thick end plates from 60×45 mm to 120×120 mm and are welded to the ends of the specimens in such a way that the centre of the plate coincided with the centroid of the specimens to facilitate the application of axial load through the centre of gravity. To facilitate the application of load axially, two plates of 16 mm thick and size $70 \text{ mm} \times 70 \text{ mm}$ with a spherical groove in between to accommodate a ball of 40 mm diameter were welded to the earlier plates such that their centers coincide. The schematic arrangement of the test set up in Olsen's Universal Testing Machine is shown in Fig. 3. Figure 4 shows the details of end condition for the stub columns loaded axially.



Fig. 3—Test set-up

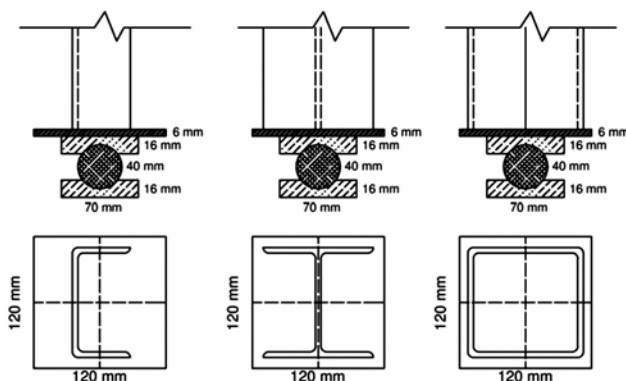


Fig. 4—Details of ball end condition

Compression tests

All the tests were conducted in the Olsen's Universal Testing Machine. Deflecto-meters having least count of 0.01 were kept to measure the vertical and horizontal deflection. One deflectometer was placed on the bending bench with its tip touching the movable head. The other five deflectometers were kept at different locations at one-third, mid and two-third height of the web, flange and the lip. For each test, the axial load was increased slowly at a rate of 0.381 mm/min an increment of one-tenth of the ultimate load in the initial stage and at closer intervals near the ultimate load. The loads were increased at an increment of one-tenth of the ultimate load in the initial stage and later at closer intervals near the ultimate. The deflectometer readings were taken at every increment of load. The load was increased further until the ultimate load was reached. This was observed visually and also by the rapid increase in the deflectometer readings. Readings were also taken beyond the ultimate load to study the post buckling strength of the specimen. The specimens were loaded to the maximum limit beyond which there was rapid increase in the dial gauge readings with no increase in load.

FE Modelling, Analysis and Calibration

An experimental investigation can be supplemented with a numerical method of predicting the behaviour within the acceptable accuracy limit. The finite element method (FEM) is a numerical technique to obtain approximate solutions to a wide variety of engineering problems where the variables are related by means of algebraic, differential and integral equations. Finite element analysis converts the physical structure to a complex system of points called nodes which make a grid called a mesh. Firstly, a numerical model of a cold-formed steel member should have the capability of representing thin to moderately thick elements with flat surface as well as curved surfaces. Secondly it should accommodate different cross-section configurations and different loading conditions and thirdly it should take into account large out of plane deformations, large rotations and elasto-plastic material behaviour. Fourthly it should represent the variation of material properties due to the variation of cross-section caused by cold forming operations and lastly it should incorporate initial geometric imperfections and out of plane movements of plate components. In view of these a non-linear finite element method may be the only feasible analysis tool for cold- formed steel

members. Cold-formed steel sections have thickness in one direction which is extremely small compared to the other two dimensions are modeled as a plate-shell element. The program has static, stability and non-linear analysis capabilities, which are used in this study. The commercial non-linear finite element analysis program ABAQUS 6.10 was used to predict the behaviour and failure modes of the channel sections at axial loading with pinned end condition.

The numerical simulation consisted of three stages. In the first stage a linear analysis was performed on a perfect geometry and secondly buckling analysis to incorporate the initial imperfections was performed with the same geometry to establish probable buckling modes of a column. The third stage, a non-linear analysis was performed incorporating material geometric imperfections to obtain the ultimate load and failure modes of the column. In the finite element model, the measured cross-section dimensions and material properties from the tests were modeled. The model was based on the center line dimensions of the cross-sections.

Element type and mesh

If the section thickness is less than $1/10$ of an elemental dimension then the section can be modeled with shell elements. Shell element would be an appropriate choice for the cold-formed steel members since their thickness is very small when compared to the width of the element. ABAQUS shell element library provides numerous types of shell elements that allow curved modeling, intersection of shells and nonlinear material response. These shell elements are divided into three categories consisting of general-purpose, thin, and thick shell elements. For performing static analysis, general-purpose shell elements are appropriate and in that 4-noded shell elements are adopted for most of the FE models. S4R a 4-noded doubly curved general-purpose shell, elements with reduced integration with hourglass control and with finite membrane strains is used as shell element. Higher resolution of the transverse shear may be obtained by stacking continuum shell elements. C3D8R a solid element is used to model the end plates which are used to distribute the load over the column section. This element is defined by 8-noded linear brick, reduced integration with hourglass control. Figure 5 shows the element characteristic of Solid C3D8R. A high mesh density usually increases the accuracy of the results obtained at the expense of computation time while low mesh density can lead to serious errors. Element aspect ratios are kept in $\frac{1}{2}$ -2

range as it was found to give better results as per literature review. Element meshes are refined until the acceptable converged solution is obtained. For numerical simulations, the specimens were considered pinned at one end and simply supported at the other end.

Material characteristics

A number of material related factors can cause the change in structure stiffness and one such factor is stress strain relationship of the material. The material behaviour is assumed as linearly-elastic/perfectly-plastic /linearly-strain hardening in non-linear analysis as shown in Fig. 6.

Boundary condition

For the unloaded end, all three translations together with the rotation along the length of the column were restrained, while the rotations about vertical and horizontal moment inertia axes were allowed. For the loaded end, the translations along section axis and the rotation about longitudinal profiles axis were restrained, while the rotations about major and minor

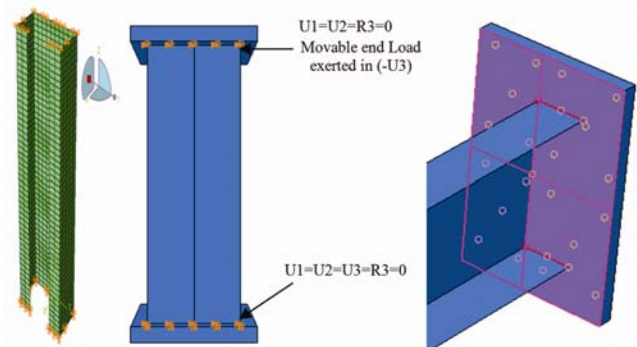


Fig. 5—Numerical model with meshing, boundary condition and tie constraint

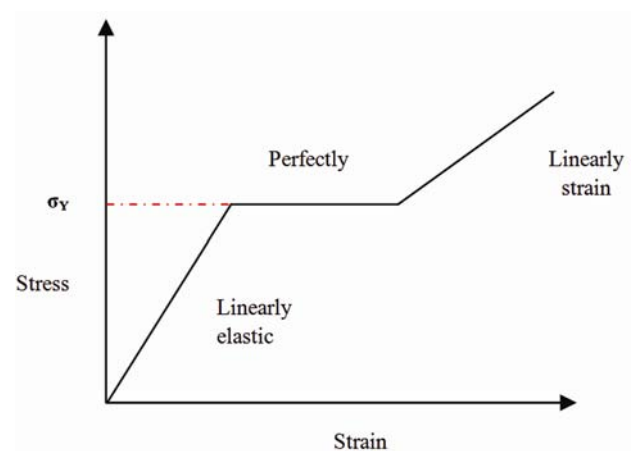


Fig. 6—Type of stress strain curve used in the model

moment inertia axes together with longitudinal translation was allowed. Residual stresses and initial plastic strains were not considered for this study. The specimens were loaded axially through the CG of the specimens. The columns are loaded in displacement control, with the displacement applied at the centroid node. Tie constraint as shown in Fig. 5 is used to connect the plate and the column ends keeping the slave element and master element as the beam and the column top end and vice versa at the bottom end.

Loading and analysis

Prior to analyzing the post-buckling behaviour the structure, linear buckling analysis is performed secondly on the specimens to obtain its buckling mode shape. The main purpose of the perturbation analysis is to establish probable buckling modes (Eigen mode) of the columns. The eigen mode is then scaled by a factor (scale factor) to obtain a perturbed mesh of the column for the nonlinear analysis. The buckling analysis is carried out to predict the buckling loads and the corresponding buckling shapes. These are used as a parameter in determining the post-buckling strength and for incorporating the input values of the geometric imperfection by using first few buckling modes shape values. The buckling analysis is carried out for the eigen values of 10 by using LANCZOS eigensolver by applying an initial displacement of 1 mm at the point of application of the load. Then the displacement corresponding to local and overall buckling is added to get the total displacement. The displacement control loading method using the modified Riks nonlinear solution algorithm is used, which is identical to the loading method used in the experiments. Nonlinear post-buckling analysis is carried out to study the load

versus end-shortening characteristic curves and ultimate load capacity. Young's modulus, E and the yield stress, f_y of the steel material were considered as per Table 2 described earlier. For each incremental step of end-shortening, the total load or reaction at the end is obtained. Schafer and Pekoz¹² suggested expression for the average degree of imperfection for the cold-formed steel members is between $0.14t$ and $0.66t$, where t is the thickness of sheet steel. Hence in this study, the results based on the Schafer and Pekoz's expression were used in finite element analyses. The local buckling imperfection amplitude of $0.25t$ and the overall imperfection amplitude of $L/1000$ were used for the axially loaded channels respectively.

Results and Discussion

This investigation presents a detailed buckling strength of an experimental, numerical and theoretical investigation of a total of twenty four cold-formed steel channels as stub columns with ends as hinged tested till failure. The behavior of cold-formed steel single channels, channels welded back to back and channels welded toe to toe as box sections were studied. Initial stiffness and ductility co-efficient was calculated for all the sections from the load versus axial behaviour of experimental and FEA analysis under elastic and also in the plastic ranges of loading were studied. The ultimate load carrying capacities of the specimens were compared with the load carrying capacities predicted using the Indian Standards, US Standards and the British Standards. The effect of lip and the effect of size of the section on the load carrying capacity and the modes of failure of the channels were also studied.

Table 2—Comparison of experimental and finite element analysis (FEA) and the codal values for single channel sections

Single Channels	Length (mm)	P_{EXP} (kN)	P_{FEA} (kN)	P_y/P_{FEA}	P_{EXP}/P_{FEA}	P_{IS}/P_{FEA}	P_{NAS}/P_{FEA}	P_{BS}/P_{FEA}
40×20×2.1	120	58.29	57.93	0.84	1.01	1.05	0.97	0.93
40×20×10×2.1	120	68.97	86.55	0.74	0.80	0.83	0.81	0.67
60×30×2.1	180	73.42	80.92	0.82	0.91	0.75	0.99	0.90
60×30×15×2.1	180	97.90	117.22	0.75	0.84	0.97	0.97	0.85
80×40×3.3	250	112.10	97.80	1.01	1.15	1.21	1.15	1.11
80×40×20×3.3	250	142.40	158.94	0.73	0.90	0.89	0.87	0.76
100×50×3.3	300	135.70	135.68	0.89	1.00	1.01	1.01	0.99
100×50×20×3.3	300	138.80	173.47	0.51	0.80	1.00	1.00	0.89
			Mean	0.79	0.92	0.96	0.97	0.89
			SD	0.24	0.21	0.14	0.10	0.27

P_{BS} : Ultimate load predicted by the British Standards; P_{EXP} Experimental ultimate load;

P_{FEA} : FEA ultimate load; P_{IS} : Ultimate load predicted by the Indian Standards; P_{NAS} : Ultimate load predicted by the US Standards;

P_y : Column yield load

Load carrying capacity

The capacity of the double channels welded back to back was approximately twice that of single channels. Similarly the load carrying capacity for the channels welded toe to toe as box sections is slightly higher than that of channels welded back to back. The ultimate load carrying capacity of double channels welded toe to toe as box sections was up to 2.5 times that of single channels. The provision of lip increased the ultimate load carrying capacity of single channels up to 63% and 33% from the FEA and experimental studies, respectively. The experimental, numerical and theoretical ultimate loads obtained for single channels and also their ratios are presented in Table 2 with their effective lengths. It was observed both theoretically and numerically that in the case of single lipped channels section the failure was initiated first

in the flange, whereas in the case of single plain channels the failure was initiated in the web.

It was observed that the predicted ultimate load carrying capacity by NAS method for single channels had a good correlation with the experimental values. The prediction was at a margin of 20% and up to 15% using IS: 1608 Part-1¹¹ method and BS: 5950 Part 5¹³ method, respectively. The ultimate load carrying capacity of single channels with largest flat width to thickness ratio was up to 1.35 times more than that of channels with the least flat width to thickness ratio. The experimental, numerical and theoretical ultimate loads obtained for channels welded back to back and also their ratios are presented in Table 3 with their effective lengths. The provision of lip increases the ultimate load carrying capacity of double channels welded back to back up to 40% and 30% from the FEA and experimental studies respectively.

Table 3—Comparison of experimental and finite element analysis (FEA) and the codal values for double channel sections welded back to back

Double channels welded back to back	Length (mm)	P_{EXP} (kN)	P_{FEA} (kN)	P_y/P_{FEA}	P_{EXP}/P_{FEA}	P_{IS}/P_{FEA}	P_{NAS}/P_{FEA}	P_{BS}/P_{FEA}
40×20×2.1	120	119.7	118.63	1.06	1.01	1.03	1.00	0.91
40×20×10×2.1	120	154.9	148.33	0.99	1.04	0.97	0.95	0.78
40×40×10×2.1	250	180.7	204.06	1.06	0.86	1.10	1.09	0.98
40×40×10×3.3	250	218	198.04	0.96	1.10	0.94	0.94	0.73
60×30×2.1	180	173.5	158.62	1.23	1.09	1.00	1.06	0.98
60×30×15×2.1	180	198.9	203.67	1.15	0.98	1.12	1.12	0.98
80×40×3.3	250	240.3	209.45	1.16	1.15	1.14	1.09	1.04
80×40×20×3.3	250	311.5	294.49	0.98	1.06	0.96	0.93	0.82
100×50×3.3	300	240.3	250.92	1.23	0.96	1.10	1.12	1.07
100×50×20×3.3	300	271.4	315.66	0.56	0.86	1.10	1.09	0.98
			Mean	1.04	1.01	1.05	1.04	0.93
			SD	0.19	0.09	0.07	0.07	0.11

P_{BS} : Ultimate load predicted by the British Standards; P_{EXP} : Experimental ultimate load;

P_{FEA} : FEA ultimate load; P_{IS} : Ultimate load predicted by the Indian Standards; P_{NAS} : Ultimate load predicted by the US Standards;

P_y : Column yield load

Table 4—Comparison of experimental and finite element analysis (FEA) and the codal values for double channel connected as box sections

Double channels connected as box section	Length (mm)	P_{EXP} (kN)	P_{FEA} (kN)	P_y/P_{FEA}	P_{EXP}/P_{FEA}	P_{IS}/P_{FEA}	P_{NAS}/P_{FEA}	P_{BS}/P_{FEA}
40×20×2.1	120	125.90	139.31	0.83	0.90	0.88	0.85	0.77
60×30×2.1	180	178.90	201.01	0.85	0.89	0.96	0.85	0.78
60×30×15×2.1	180	198.00	222.89	0.72	0.89	1.03	1.03	0.90
80×40×3.3	250	244.70	262.35	0.79	0.93	0.91	0.87	0.83
80×40×20×3.3	250	302.60	305.20	0.76	0.99	0.93	0.90	0.80
100×50×3.3	300	195.80	297.12	0.63	0.66	1.02	0.95	0.88
			Mean	0.76	0.88	0.96	0.91	0.83
			SD	0.10	0.12	0.06	0.07	0.06

P_{BS} : Ultimate load predicted by the British Standards; P_{EXP} : Experimental ultimate load;

P_{FEA} : FEA ultimate load; P_{IS} : Ultimate load predicted by the Indian Standards; P_{NAS} : Ultimate load predicted by the US Standards;

P_y : Column yield load

The prediction of ultimate loads carrying capacity by NAS¹⁰ and IS method for the double channel welded back to back is in good correlation with the FEA and experimental results. Whereas the prediction by BS: 5950 Part-5¹³ method underestimates the FEA results up to 9% except for CFC 80×40×3.3 mm and CFC 100×50×3.3 mm which overestimates the FEA results up to 7%. The load carrying capacity of the section with largest flat width to thickness ratio is 1.11 times more than the specimen with least flat width to thickness ratio. The experimental, numerical and theoretical ultimate loads obtained for channels welded toe to toe as box sections and also their ratios is presented in Table 4 with their effective length. The provision of lip increases the ultimate load carrying capacity of double channels welded back to back up to 24% and 16% from the FEA and experimental studies, respectively.

The prediction of ultimate load carrying capacity by IS method for the double channel welded toe to toe as box section is in good correlation with the FEA and experimental results, whereas the prediction by BS:5950 Part-5¹³ method underestimates the FEA results up to 23%. The percentage of underestimation by IS and NAS method with respect to the FEA results are up to 15% and the percentage of overestimation is 3%. For all the specimens the prediction by the British standards is the least.

It is observed that in case of channels welded back to back or as box sections the increase in ultimate load is twice when compared with single channels irrespective of cross-section. The predicted ultimate load is higher than the observed ultimate load by a margin of 15%. The observed ultimate load carrying capacity increased by 20% for sections with lip as compared with sections without lip. The buckling coefficient, k , recommended by US Standards does not account for compressive stress gradient and the k value corresponding to uniform compression has been prescribed for all stress gradients. NAS Manual¹⁰ does not account for the increase in the buckling strength due to edge rotational restraint provided by the adjoining elements. These lead to underestimation of the local buckling strength and hence the ultimate strength is greatly reduced while loading the specimens eccentrically. In the case British Standards the underestimation of the local buckling strength results in the corresponding underestimation of the effective widths in the post-local buckling range. The effective area of the section is determined at the yield

stress stage rather at the actual failure stage has lead to further underestimation of the ultimate loads. Indian Standards does not allow unstiffened elements to be stressed beyond the local buckling strength under uniform compression, thus disregarding the appreciable post-local buckling strength. The local buckling strength and the post buckling behaviour do not account for stress gradient resulting in conservative ultimate strength when the eccentricities cause larger compression at the supported edge of the unstiffened elements.

Load versus axial shortening

The load versus axial shortening behaviour is initially linear in all the cases up to 75% of ultimate load irrespective of the cross-section of the specimens. As the end-shortening increases and exceed the critical buckling load, the columns enter the post-buckling range where non-linear behaviour dominates. In the post buckling range the load increases progressively at a slower rate and the end-shortening increases rapidly. At the ultimate load, the stub columns failed and lost its ability to carry further loading. Therefore, the load curves decreases with the end-shortening beyond the ultimate load. Figures 7-9 show the load versus axial shortening behaviour of

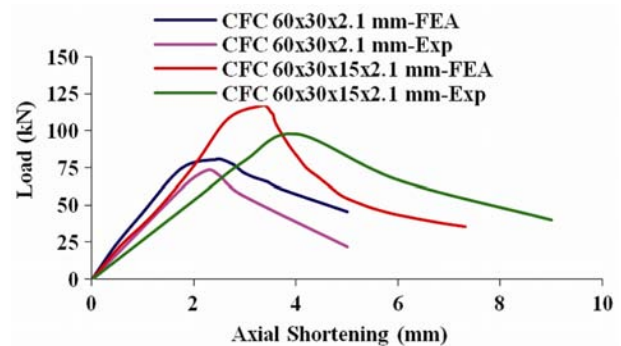


Fig. 7—Axial shortening behaviour of single channels

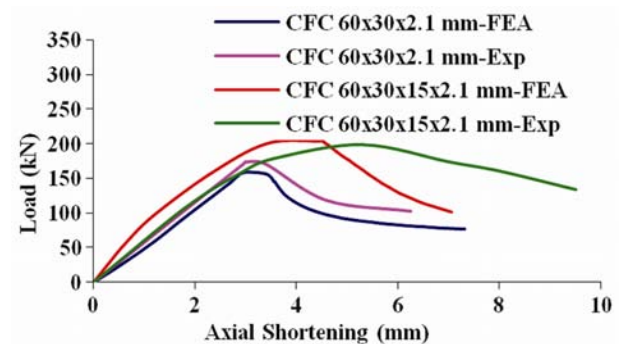


Fig. 8—Axial shortening behaviour of double channels welded back to back

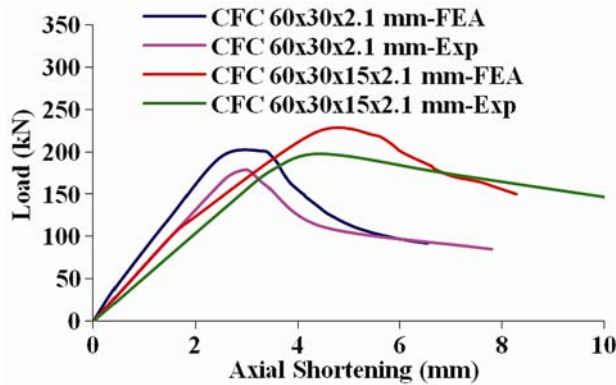


Fig. 9—Axial shortening behaviour of double channels welded toe to toe as box sections

experimental and FEA results for single channels, double channels welded back to back and double channels welded toe to toe both with and without lip. It was observed that the behaviour is steeper for channels without lips as compared to channels with lip. Similarly for channels without lips the drop in the load after it reaches the ultimate load is sudden while the drop is gradual for sections with lips. The long horizontal plateau after the ultimate load is reached for channels with lips indicates high degree of ductility.

Load versus lateral deflection

The lateral deflection behaviour for the lipped channels is shown in Fig. 10. The lateral deflection behaviour of single channels, channels welded back to back and channels welded as box sections were meager upto ultimate load and beyond which the deflections were very large. For the channel sections without lips the lateral deflection corresponding to the maximum load carrying capacity at flanges is slightly higher than that of deflection at webs for all cross-sections. Whereas the maximum deflection for lipped channels of single sections is twice than that of double and box sections. It was observed that in the case of lipped single and double open channels, the mode of failure was first initiated in the flange, whereas in the case of single channel sections without lip, the failure was initiated in the web. Channels welded as box sections with lips, the failure occurred always in the web which are stiffened by the flanges.

The initial stiffness was calculated for all the specimens observed from the test results and FEA studies, from the load versus axial shortening behaviour. In the case of single channels the initial stiffness is 15% higher for sections having larger flat

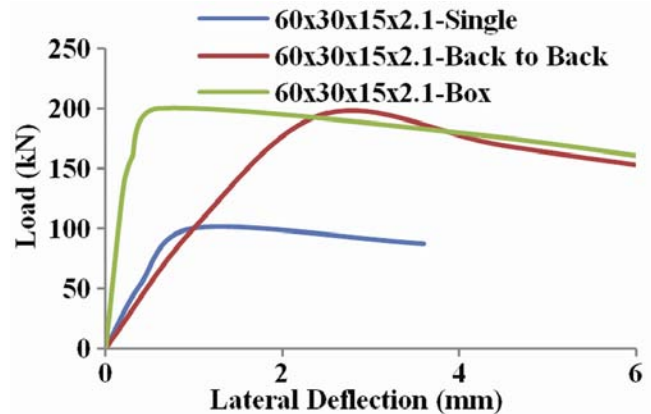


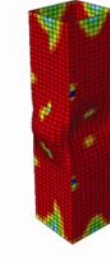
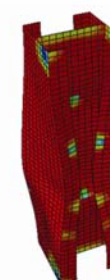
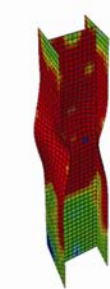
Fig. 10—Load versus lateral deflection of lipped channels

width to thickness ratio. The stiffness of lipped channels is 10% lower compared with plain channels for sections with lesser flat width to thickness ratio. Compared to single plain channels and channels welded toe to toe as box sections, the stiffness for channels welded back to back is over 1.5 times and about 1.8 times more, respectively. Except for few sections the initial stiffness is higher in FEA studies compared with experimental results. Maximum stiffness is observed for double channels welded back to back and welded toe to toe as box sections in the FE analysis.

From the load versus axial shortening behaviour of the single channels, channels welded back to back and channels welded toe to toe as box sections, the ductility co-efficient is calculated which was defined by Han *et al.*¹⁴ It is calculated as the ratio of the axial displacement (Δu) corresponding to 85% of ultimate load in the post ultimate region to the axial displacement at yield load (Δ_y). For lipped channels welded as back to back sections, the co-efficient is up to 50% more as compared with lipped single and channels welded as box sections. It is observed that the ductility co-efficient predicted by both the experiments and FEA, is the highest for channels welded back to back for both lipped and unlipped sections. The difference between the co-efficients predicted by FEA and the experiments is lesser for channels welded toe to toe as box sections whereas the difference is upto 50% more for others.

Table 5 shows the deformed shapes of the finite element analysis and experimental tested specimens for the single channels, double channels welded back to back and the double channels welded as box sections respectively. Thus the prediction by finite element analysis is the same as the experimental results.

Table 5—Experimental and FEA investigation of the columns

Specimen ID	Failure Mode	Location of Failure	Experimental	FEA
60x30x2.1 mm	Local	On third height of the column, the flanges distorted in wards		
60x30x2.1 mm	Overall Buckling	Web failure with outward buckling		
80x40x20x3.3 mm	Local + Overall Buckling	Inward buckling of lips at one third height of the column		
100x50x3.3 mm	Local	At mid height of the column, the flanges distorted inwards		

Modes of Failure

Singly symmetric channel sections with lip fail by overall buckling which occurs between one third and mid height. Singly symmetric channel section without lip fails by local buckling of the flange, which occurs between one third and mid height of the specimen. Doubly symmetric channel section (channel section welded back to back) with lips fails by overall buckling and plate buckling, which occurs at one-third height. Doubly symmetric channel

section (channel section welded flange to flange) without lip fails due to element buckling. Doubly symmetric square channels section with lip welded back to back fails by overall and plate buckling, which occurs between one third and mid height. Doubly symmetric box section with inside lip fails by overall buckling of web where there is no lip. Doubly symmetric section (channel sections welded back to back) with lips fails by plate and overall buckling.

Conclusions

From the series of tests conducted on single channel, channels welded back to back and box sections with and without lip; the following conclusions are drawn:

- (i) The vertical and lateral deformations were small up to failure and beyond the ultimate load the deformations were substantial.
- (ii) The predicted ultimate load obtained by finite element method is up to 52% higher for the channel section 100×50×3.3 mm, the lesser experimental load observed was due to local buckling of the end plate observed during the loading process. The predicted ultimate load carrying capacity increases up to 38% for sections with lip as compared with sections without lip for single channel sections.
- (iii) All the stub columns start to behave non-linearity when the stress is larger than one-third of the yield stress of the material. For the channels welded back to back the increase in the predicted ultimate load by the finite element analysis is up to 29%. The increase in the predicted load carrying capacity obtained from the FEA for the channels connected as box sections are up to 32%.
- (iv) Singly symmetric channel sections with lip fails by overall buckling which occurs between one third and mid height. Singly symmetric channel section without lip fails by local buckling of the flange, which occurs between one third and mid height of the specimen.
- (v) Doubly symmetric channel section (channel section welded flange to flange) without lip fails due to local buckling. Doubly symmetric square channels section with lip welded back to back fail by overall and local buckling, which occurs between one third and mid height. Doubly

symmetric section (channel sections welded back to back) with lips fails by plate and overall buckling.

- (vi) Doubly symmetric box section with inside lip fails by overall buckling of web where there is no lip.
- (vii) The provision of lip increases the load carrying capacity almost twice for the single plain channel section. The singly symmetric plain channels sections failed by local buckling of the flanges. Whereas lipped channels failed by a combination of local buckling plus overall buckling.

References

- 1 Anil Kumar M V & Kalyanaraman V, *J Struct Eng*, 136 (7) (2010) 879-885.
- 2 Beulah Gnana Ananthi G, Palani G S & Nagesh R Iyer, *Latin Am J Solids Struct*, 12(1) (2015) 1-17.
- 3 Jia-Hui Zhang & Ben Young, *Thin-Walled Struct*, 89 (2015) 178-191.
- 4 Stone TA, LaBoube R A, *Thin-Walled Struct*, 43 (2005) 1805-17.
- 5 Ting T C H & Lau H H, *Adv Mater Res*, 201-203(2011) 2900-2903.
- 6 Vishuvardhan S & Samuel Knight G M, *J Civil Eng Res Practice*, 2(2) (2005) 25-37.
- 7 Whittle J & Ramseyer C, *Thin-Walled Struct*, 47 (2009) 190-201.
- 8 Young B, *J Struct Eng*, 134 (2008) 727.
- 9 IS: 1608 Part-1, 'Metallic materials-Tensile Testing at Ambient Temperature', *Bureau of Indian Standards*, 2005.
- 10 NAS Manual, *American Iron and Steel Institute*, 2007.
- 11 IS: 801, Code of Practice for Use of cold-formed light gauge steel structural members in general building construction. *Bureau of Indian Standards*, 1975.
- 12 Schafer B W & Pekoz T, *J Construct Steel Res*, 47 (1998) 193-210.
- 13 BS : 5950—Part 5, "Structural Use of Steelwork in Building—Code of Practice for Design of Cold-formed Sections". *British Standards Institution*, 1987.
- 14 Han L H, Zhao X L & Tao Z, *Steel Compos Struct*, 1(1) (2001) 51-74.