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STUDY ON THE EFFECT OF SPACERS ON THE ULTIMATE CAPACITY OF INTERMEDIATE LENGTH THIN WALLED SECTION UNDER COMPRESSION*

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Abstract– This paper reports the results of experimental, analytical and numerical studies on the effect of spacers on the behaviour and ultimate capacity of intermediate length Cold-formed steel (CFS) open section column. A channel section is considered. Totally, four columns were experimented with hinged-hinged end condition. The section properties and selection of column length were obtained by performing elastic buckling analysis using CUFSM software. Finite Element models incorporating the geometric, material non linearities and initial geometric imperfection of the specimens were developed by using ANSYS and its accuracy was verified using the experimental results. Following the verification, a finite element parametric study was carried out by varying the depth and number of spacers. Experimental and numerical strength of open sections were compared with the predicted resistance by DSM – AISI S100:2007, AS/NZS: 4600- 2005, Eurocode 3 and IS: 801-1975. Effects of spacers on the ultimate load capacity of the column have been examined. The results are presented in the form of design charts. It is concluded that depth and number of spacers have significant influence on the behaviour and strength of the columns. Based on the nonlinear regression analysis the design equation was proposed for the selected section.

Keywords– Cold-formed steel, columns, distortional buckling, spacers, thin walled members

1. INTRODUCTION

The use of cold-formed steel structural members has increased rapidly in recent years. As compared to thicker hot-rolled members, cold-formed members provide enhanced strength to weight ratio and ease of construction. The manufacturing process of fabricating cold-formed members usually involves brake-pressing and roll-forming of steel sheets and strips to produce a wide range of cross-section shapes. Cold formed sections are normally thinner than hot-rolled sections and have a different forming process. Therefore, the buckling and material behaviour can be quite different. The open compression members may fail in local, distortional, flexural, and flexural-torsional buckling. Local modes and global modes (i.e. flexural and flexural-torsional buckling) are largely covered in the main design codes by means of effective widths of the plate elements and by column design equations for global buckling. Interaction of local and global modes is also considered in these codes. Distortional buckling plays an important role in the use of open cold formed steel columns. The present Indian code of practice for design of cold formed steel members, is under revision. Local and global buckling occurs at relatively short and long half wave lengths respectively. But distortional buckling occurs at intermediate half wave length.

Takahashi [1] was the first to publish a paper describing distortion of the thin walled open section. Hancock [2] presented a detailed study of a range of buckling modes (Local, distortions, and flexural-

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torsional) in a lipped channel section. Lau and Hancock [3] provided simple analytical expressions to allow the distortional buckling stress to be calculated explicitly for any geometry of cross-section of thin-walled lipped-channel section columns. Kwon and Hancock [4] studied simple lipped channels and lipped channels with intermediate stiffener under fixed boundary conditions. They chose section geometry and yield strength of steel to ensure that a substantial post-buckling strength reserve occurs in a distortional mode for the test section. Lau and Hancock [5] provided design curves for sections where the distortional buckling stress and yield stress were approximately equal. Davies and Jiang [6] used the Generalized Beam Theory to analyse the individual buckling modes either separately or in selected combinations.

Schafer and Pekoz [7] have studied the effect of geometric imperfections and residual stresses on computational modelling of cold-formed steel columns. Distortional buckling strength of a few innovative and complex geometrical sections has been studied by Narayanan and Mahendran [8]. For intermediate length pallet rack columns, the distortional strength was studied by providing spacers to connect the flanges of upright sections by Talikoti and Bajoria [9]. The partly closed thin walled steel columns were studied by Milan Veljkovic and Bernt Johanson [10]. Kwon et al. [11] studied the buckling interaction of the channel columns. Anil Kumar and Kalyanaraman [12] studied the evaluation of direct strength method for CFS Compression members without stiffeners. Showkati and Shayadeh [13] have studied the behaviour of steel frames investigated under an applicable range of initial rotational geometric imperfection in joints. Anbarasu and Sukumar [14] studied the connectors interaction on behaviour and ultimate strength of stiffened channel columns.

In conclusion, the existing research results show that the effect of spacers on the buckling strength of members under axial compression is less pronounced for ordinary strength steel. So the distortional buckling strength of members under axial compression may be enhanced. Furthermore, the existing research results are all based on the regular cross sections. That is to say, there is a lack of study on the distortional buckling behaviour of complex stiffener members with spacers under axial compression, which makes it necessary to perform a systematic study with both experimental and numerical investigations.

Open cold-formed steel sections such as C, Z, hat and rack sections are relatively common because of their simple forming procedures and easy connections, but they suffer from certain buckling modes due to their mono-symmetric or point symmetric nature, high plate slenderness, eccentricity of shear centre to centroid and low torsional rigidity. The ultimate capacity of axially loaded intermediate length CFS channels may adversely be affected by distortional buckling. To either eliminate or delay the distortional buckling mode, the spacers are introduced. The main aim is to examine the behaviour and strength of intermediate length pin-ended column with spacers for which there are no design rules currently available. Spacers are the transverse elements of CFS sheet used to connect the lips of the open sections using self-drilling screw. For this work, channel section with complex stiffeners was considered. A finite element model simulating the behaviour of this section was developed. Experimental results were used to verify the efficiency of the numerical model in predicting ultimate capacities and the corresponding response of the tested specimens. Totally four columns were tested (one full open and the others with spacers from 1 to 3). A parametric study was performed to investigate the effect of spacers on the behaviour and strength of this section by varying the depth and number of spacers. The test and numerical results for the full open sections are compared with design strength calculated using the DSM – AISI 100:2007, AS/NZS: 4600-2005 and IS: 801-1975 for cold-formed steel structures.

2. TEST PROGRAM

In total, 4 numbers of columns were fabricated with 1st fully opened, 2nd one spacer, 3rd two spacers and 4th three spacers. The specimens were tested under axial compression. Preliminary investigations were carried out to find suitable test section type, thickness and steel grade of light gauge cold formed steel commonly used for structural members. These sections were analysed using CUFSM to find the most suitable geometry of the specimen. The length of the specimen was selected as a multiple of buckling half wave lengths from CUFSM analysis.

a) Test specimens

A channel section with complex stiffener was chosen for the study. The dimensions of the cross section are finalized keeping the plate slenderness ratio (b/t) within limits to eliminate local buckling.

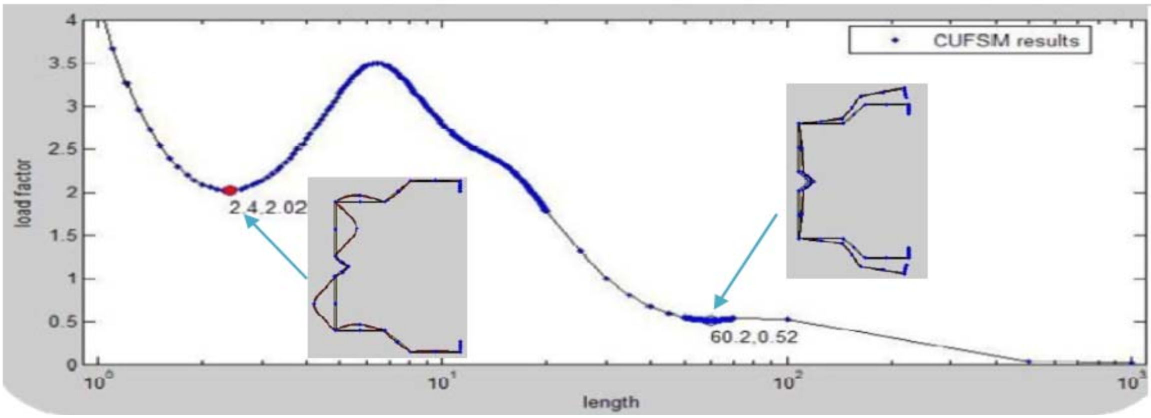

Fig. 1. Buckling plot of a selected section

The ends of the columns are considered as pinned. Figure 1 shows the typical buckling plot of RC-S0 specimen obtained from CUFSM software. The buckle half wave length shown in x axis is in the unit of inches. From the Fig. 1, it can be observed that distortional buckling occurred at an intermediate half wavelength of approximately 1529 mm and also the rotation of the sides about the uppermost flange web junction. Hence the length of the column is chosen as1600 mm. Distortional buckling may govern the intermediate length column strength of open sections and often interacts with local buckling depending upon the section geometry. The chosen section nominal dimensions and cross-section profile is shown in Fig. 2.

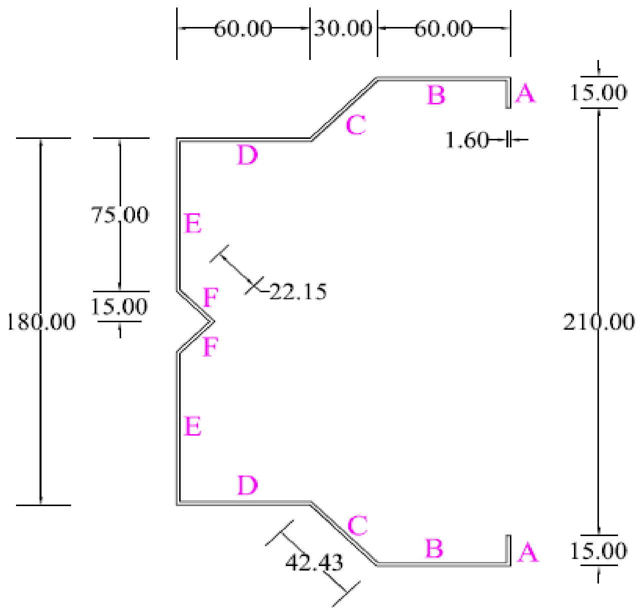

Fig. 2. Cross section of rack column

The specimens are fabricated by press braking operation. Spacers are the transverse elements made up of the same material used for specimen and are cut into the required shape and connected to the lips of the section using self-drilling screw. The self drilling screw can provide a rapid and effective means to fasten CFS sheet metal spacers to members than any other conventional methods of connections. The self-drilling screws have the ability to drill their own hole and form, or tap their own internal threads without deforming their own thread. One screw of 6mm in diameter is used at each interconnection. Overall section dimensions were measured for each test specimen, from which centreline dimensions of specimen cross-section were calculated. Table 1 presents the values of the measured dimensions of test specimens.

Table 1. Measured cross-section dimensions of Test Specimens

Specimen ID	Dimensions in mm						Number of spacers	Average thickness
	A	B	C	D	E	F		
RC-S0-d50	15.1	61.6	41.4	60.3	75.3	22.5	0	1.6
RC-S1-d50	15.3	62.3	42.3	60.5	75.2	22.5	1	1.6
RC-S2-d50	15.2	62.5	42.5	60.4	75.2	22.5	2	1.6
RC-S3-d50	15.3	63.2	43.1	60.4	75.3	22.5	3	1.6

b) Labelling

The test specimens were labelled such that the type of column, number of spacers, and depth of spacer were expressed by the label. For example, the label "RC-S0-d50" defines the following specimen:

- "RC" indicates that the section is a rack column;
- "S0" indicates the number of spacers = zero (No spacer) alternatively "S1" indicates number of spacers equal to one,
- "d50" indicates the depth of spacer = 50mm alternatively "d75" indicates depth of spacer = 75mm.

c) Tension coupon tests

Tensile coupons were prepared and tested according to IS 1608-2005 (Part-1) to determine the yield stress (σ_y), tensile strength (σ_u), initial tangent modulus (E) and percentage elongation after fracture (e_u). The measured values are reported below.

Yield Stress (σ_y)	= 350Mpa	Ultimate Stress (σ_u)	= 450Mpa
Young's Modulus (E)	= 201GPa	Tangent Modulus (E_t)	= 20.12 Gpa
Poisson's Ratio (m)	= 0.3	% of elongation	= 27%

d) Test configuration

The compression tests were carried out using the 400kN capacity loading frame. The specimens had a 15mm thick rectangular steel plate welded to each end. The specimens are mounted between the platens and its verticality is checked. At either end between the platens and the end plates of the specimen rubber gaskets were placed to facilitate the pinned end condition at either support. The pin-ended bearings were designated to allow rotation about both major and minor axis, while rotations about the perpendicular axis and twist rotations were restricted, as shown in Fig. 3.

A force control method was used to apply a uniformly distributed compression load gradually. Three LVDTs (Linear Variable Displacement Transducers) were used to measure the deformations of specimens during testing. First, LVDT was used to measure the out-of-plane deformation at the centre of the web; secondly, LVDTs were used to measure the out-of-plane deformation of flanges. Axial shortening of the specimens was also recorded during the tests. The centroid axis of the end plates and the specimen cross section were kept the same to simulate a uniformly distributed load for pin-end conditions. Further, the two end plates were kept parallel to each other and their horizontal level was maintained to ensure a uniformly distributed load. In this way, the compression load was applied to the centroid of the column without any moments. The test configuration is shown in Fig. 3. The lateral and axial deformations of the column were recorded for every increment of load. The ultimate load at which the deflection increased without increase of the load is also recorded.

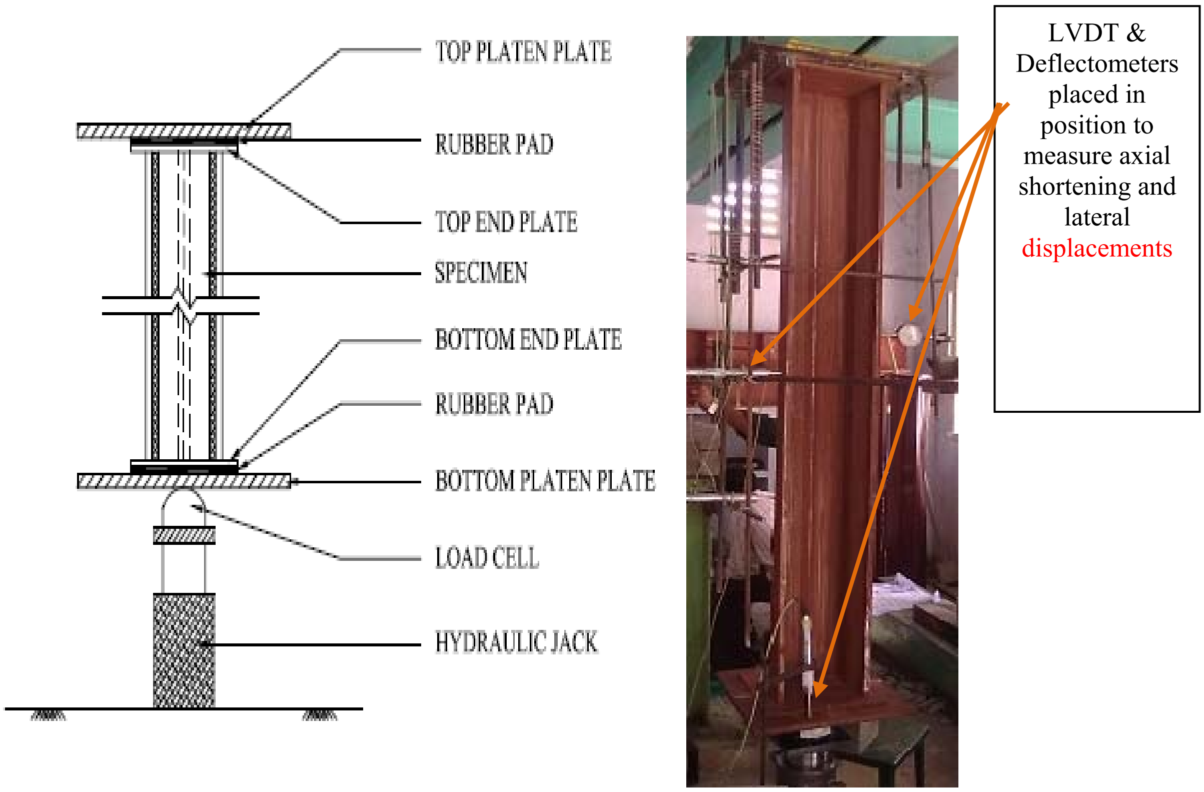

Fig. 3. Test configuration

e) Test results

The test results are presented in Table 2 as the buckling mode and ultimate load for each specimen. The final failure shapes of the intermediate length open column tested were mainly in the distortional mode which interact with flexural torsional buckling. The deformed shape of RC-S0 section is shown in Fig. 4. Also, the failure mode changes from combined distortional mode (D) and flexural torsional buckling (FT) to the interference of combined local (L), flexural buckling (F) and distortional buckling (D) mode due to the provision of spacers. The provision of spacers improves the torsional rigidity.

Fig. 4. Tested specimens

The ultimate stress σ_u of all the specimens, which is obtained by dividing the ultimate load P_u by the section area A, and the comparison with the measured yield strength σ_y are shown in Table 2.

Table 2. Test results

Specimen ID	Experimental load, KN	Failure mode	Ultimate stress, σ_u N/mm^2	σ_u/σ_y
RC-S0	130	D+FT	149.2	0.426
RC-S1-d50	138	L+D+F	158.4	0.452
RC-S2-d50	143	L+D+F	164.2	0.469
RC-S3-d50	148	L+D+F	169.9	0.485

3. FINITE ELEMENT ANALYSIS

Numerical models were created using the general purpose finite element software ANSYS and are validated on the basis of a selection of test results. The models were based on the centre line dimensions of the cross-sections. The residual stresses and the rounded corners of the sections were not included in the model. The effect of residual stresses on the ultimate load is considered to be negligible as recommended by Schafer and Pekoz [7]. Element SHELL 181, which supports nonlinear buckling analysis is used to develop the proposed finite element model. It is a 4-node element with 6 degrees of freedom at each node: translations in the x, y, and z directions, and rotations about the x, y, and z-axes are as shown in Fig. 5. In general, it is highly appropriate in linear, large rotation and large strain nonlinear analysis.

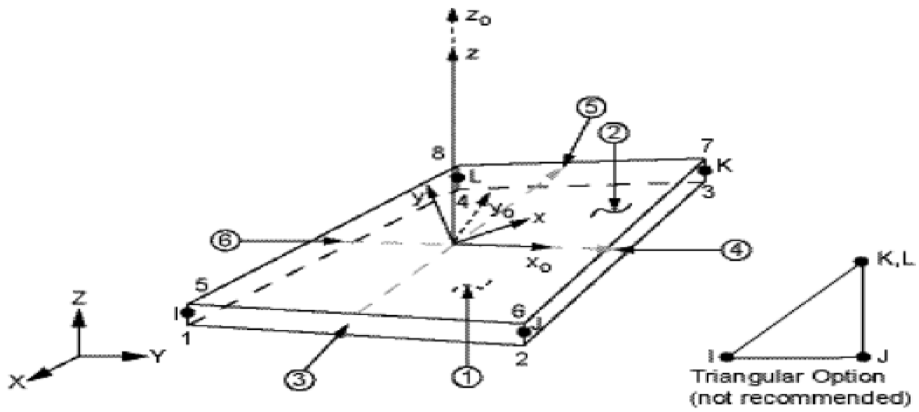

Fig. 5. Element SHELL 181

From the series of convergence studies, appropriate mesh size was chosen for this study. A linear buckling analysis was performed first to obtain the local and distortional buckling modes. The elastic modulus (E) was taken as 201000 N/mm^2.

The strain hardening of the corners due to cold forming is neglected. The boundary condition for both ends of the model is assigned to be an ideal pin end. At the unloaded end, three translational degrees of freedom are restrained as well as the rotation about the longitudinal axis. The loaded end is restrained the same as that of the unloaded end except for the translation in the longitudinal direction. A rigid surface was modeled in the loaded end. The load was applied in increment through the master node which is modeled at the centroid of the section. Two types of analysis were carried out. The first is eigenvalue analysis to determine the buckling modes and load, whereas the second is a non-linear analysis. The yield stress of the material (**σ_y**) is considered 350 N/mm^2. In order to account for the Elasto-plastic behaviour, a bilinear stress-strain curve is adopted, having a tangent modulus (E_t) of 20120 N/mm^2.

The material and geometric nonlinearity are included in the finite element model. In the nonlinear analysis, initial geometric imperfections are modeled by providing initial out-of-plane deflections to the model. The local and distortional buckling modes are extracted from the linear buckling analysis. The mode shapes are scaled to percentage of thickness and are used to create the geometric imperfections for the non-linear analysis. The maximum value of distortional imperfection was taken equal to the plate thickness as recommended by Schafer and Pekoz [7]. Local buckling imperfection was taken as 0.25 times the thickness. Since Kwon and Hancock [4] found that the overall imperfections had little effect on the buckling of the columns of intermediate length, this study does not include any overall imperfections in finite element modelling.

The FEA procedure includes 3 steps. Firstly, the finite element model without any geometric imperfection or residual stress is created, and both the displacement restraints and the axial compressive force are applied. Then a static solution is done to obtain the stiffness matrix of the model. Secondly, eigenvalue buckling analysis is conducted to obtain the buckling mode which is adopted as the initial geometric imperfection of the models in the subsequent nonlinear buckling analyses. Thirdly, after incorporating the geometric imperfections a nonlinear buckling analysis is carried out to obtain the ultimate load carrying capacity of the model.

4. VERIFICATION OF FINITE ELEMENT MODEL

A comparison between the FEA results and the test results are shown in Table 3, where P_{test} denotes the test results and P_{FEA} denotes the FEA results. From Table 3 it is shown that the FEA results agree reasonably well with the test results and the difference is slight. The mean and standard deviation of the Test to FEA ultimate loads are 0.976 and 0.0022 respectively. As an example, the load deflection curve obtained in FEA is compared with the test results for RC-S0 section in Fig. 6 and closely matches the experimental results.

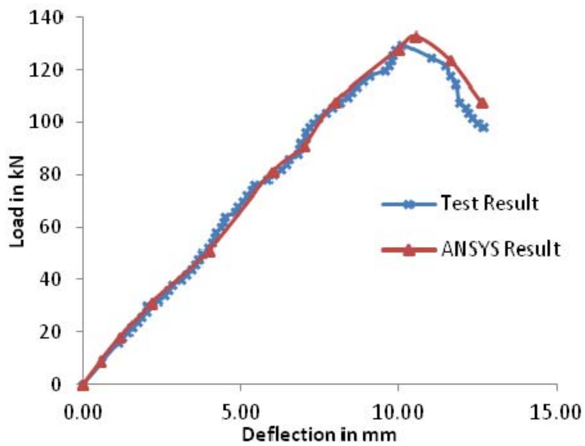

Fig. 6. Comparison of results

Table 3. Comparison of results

Specimen ID	P_{EXP} in kN	P_{FEA} in kN	P_{EXP} P_{FEA}
RC-S0	130	133	0.966
RC-S1-d50	138	141	0.973
RC-S2-d50	143	146.6	0.979
RC-S3-d50	148	152	0.979
Mean			0.976
Standard deviation			0.0022

The shape of the failure mechanism obtained in FEA was very close to the experimentally obtained shape. Figure 7 shows the deformed shape of the RC-S0 column obtained experimentally and verified by the finite element model. As can be seen, very good agreement was achieved. Similar results were obtained for other specimens also. Thus, the proposed finite element models are able to analyze the strength and buckling behaviour of columns under axial compression after incorporating the initial geometric imperfections and they are readily applicable for further parametric studies.

Fig. 7. Deformed shape of RC-S0

5. PARAMETRIC STUDY

The verification study showed that the finite element model is capable of predicting the behaviour of columns with sufficient accuracy. Hence, the finite element method was employed to investigate the effects of using the spacers on the ultimate load carrying capacity of steel column.

After the validation of the finite element model, a series of finite element parametric study is carried out to examine the effect of the depth and number of spacers on the ultimate load of the columns under axial compression. There are several parameters that have direct influence on the response/behaviour of the column. The parameters are illustrated in Fig. 8.

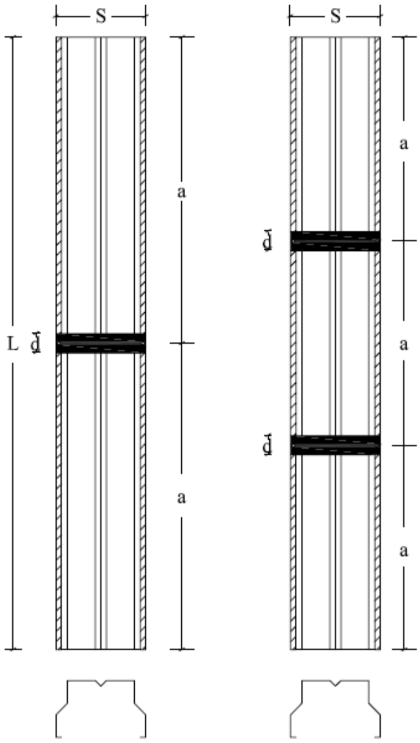

Fig. 8. Geometry of the columns

- The term 'λ_s' which is defined as the spacer plate slenderness ratio and is given by,

$$\lambda_s = \frac{d}{t}\sqrt{\frac{\sigma_y}{E}}$$

Where, d = depth of spacer plate
t = thickness of spacer plate

- The term 'a/L' which is defined as the ratio of the center to center distance (a) between spacers-to-the overall length (L) of the Column
- The term 'd/S' which is defined as the ratio of the depth of the spacer plate-to-the breadth of the spacer plate

The present study varies the depth of the spacer (d) as 50 mm, 75 mm & 100 mm. For each depth of spacer (d), the number of spacers varied from 1 to 5. The width of the spacer is 240 mm. Hence three groups of sections are formulated based on the 'd/S' ratio. They are 0.208, 0.312, & 0.417. Since the number of spacers varies from 1 to 5, the 'a/L' ratio varies from 0.50, 0.33, 0.25, 0.20 & 0.167. The Slenderness ratio of the column is 30.66. The combination of the number of spacers and the depth of spacers are shown schematically in Fig. 9.

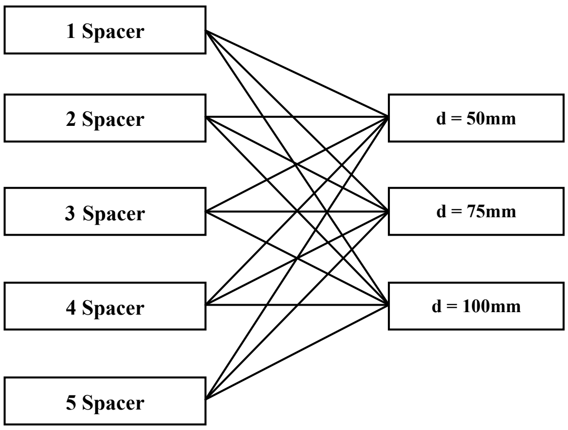

Fig. 9. Schematic representation of parametric study

6. NUMERICAL RESULTS

The normalized ratios of the ultimate-to-the yield stress of the column were influenced by various parameters and are graphically represented in the following subsections.

a) Influence of depth of spacers (d) and centre to centre length between spacers (a) on the column strength

Figure 10 demonstrates the relationship between the normalized ratio of the centre to centre length between spacers to the overall length of the Column (a/L), and the normalized ratio of the ultimate stress to the yield stress of the column (σ_u/σ_y) for different values of (d/S). Obviously, the centre to centre length between spacers has a significant effect on the strength of columns. Enhanced column strength values were obtained upon decreasing the spacers spacing (a). Increasing the number of spacers from 1 to 4 improved the ultimate strength of the columns.

b) Influence of centre to centre length between spacers-to-the overall length of the column (a/L)

Figure 11 demonstrates the influence of changing the depth of the spacer plate on the column strength for a slenderness ratio of 30.66. Apparently, as shown in Fig. 11 the column strength increases with the increase in depth of the spacer plate.

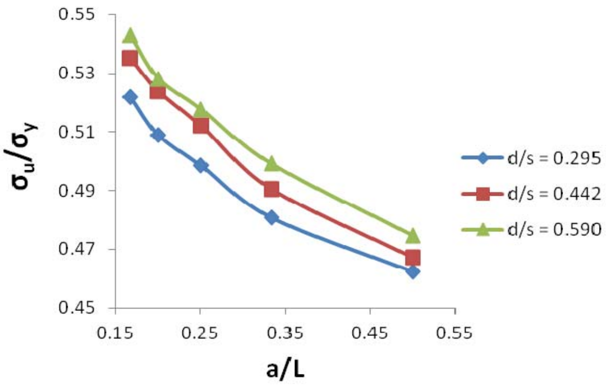

Fig. 10 – σ_u/σ_y Vs a/L

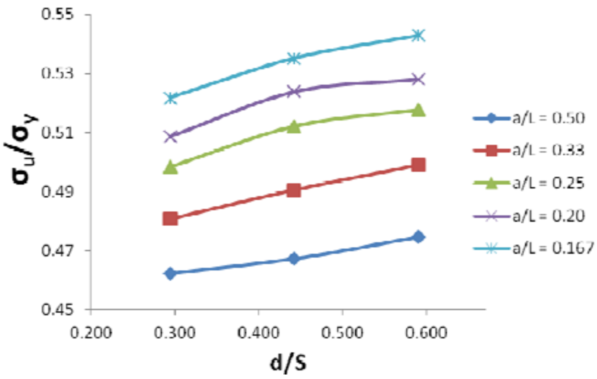

Fig. 11 – σ_u/σ_y Vs d/S

c) Influence of spacer plate slenderness ratio (λ_s)

Figure 12 demonstrates the influence of the spacer plate slenderness ratio of the column strength for a slenderness ratio of 30.66. Apparently, as shown in Fig. 12 the column strength increases with the increase in the plate slenderness ratio. The percentage increase in ultimate load of the column while varying the depth and number of spacers is shown in Table 4.

Table. 4 Increase in load carrying capacity of the column

Specimen ID	Load in kN	% increase
RC – S0	133	---
RC – S1 – d50	141	6.02
RC – S2 – d50	146.6	10.23
RC – S3 – d50	152	14.29
RC – S4 – d50	155.15	16.65
RC – S5 – d50	159.12	19.64
RC – S1 – d75	142.5	7.14
RC – S2 – d75	149.61	12.49
RC – S3 – d75	156.19	17.44
RC – S4 – d75	159.77	20.13
RC – S5 – d75	163.21	22.71
RC – S1 – d100	144.71	8.80
RC – S2 – d100	152.19	14.43
RC – S3 – d100	157.9	18.72
RC – S4 – d100	161.06	21.10
RC – S5 – d100	165.58	24.50

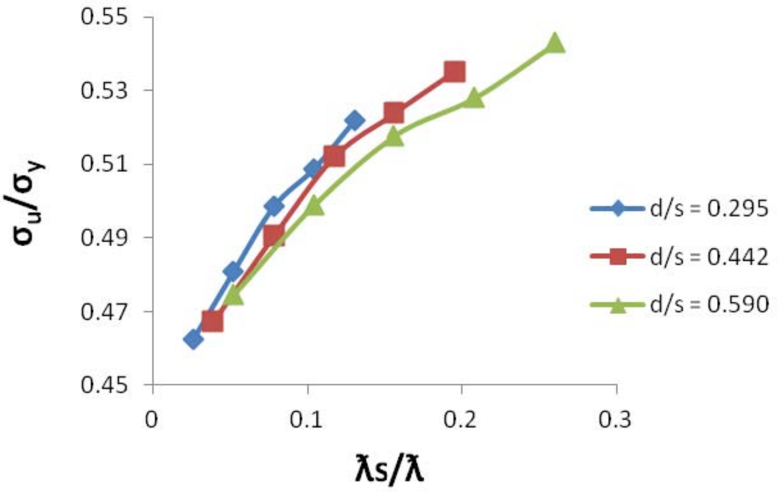

Fig. 12. σ_u/σ_y Vs λ_s/λ

7. THEORETICAL ANALYSIS

The unfactored design column strengths were calculated for fully opened section using the Indian standards (IS: 801-1975) [15], Direct Strength Method by North American Specifications (AISI S100-2007)[16], Australian/ New Zealand Standards (AS/NZS 4600:2005)[17], Eurocode 3 (EN 1993-1-1 2005) [18]. The design procedure of calculating the ultimate capacity of single symmetric compression members is fully described in corresponding codes [15-18].

The comparison of the unfactored design strengths predicted using the Indian standards (IS: 801-1975), Direct Strength Method by North American Specifications (AISI S100-2007), Australian/ New Zealand Standards (AS/NZS 4600:2005) and Eurocode 3 (EN 1993-1-1 2005) are shown in Table 5.

Table 5. Theoretical analysis results

Specimen	Ultimate load in kN				
	P_{Exp}	P_{IS}	P_{DSM}	$P_{AS/NZ}$	P_{EURO}
RC - S0	130	234.15	134.85	128.68	122.21

The Direct Strength Method (DSM) eliminates the tedious calculations of effective width of each plate element in the section. This method is now recommended as an alternative method by AS/NZS 4600 and NAS. The major advantage of this method is its ability to calculate the capacity of members subjected to local buckling interacting with other buckling modes. However, the direct strength method needs accurate elastic buckling loads based on a rational buckling analysis to calculate the ultimate capacity. Indian Standard specification (IS 801:1975) for calculating the load carrying capacity of the axial compression member is based on the effective width method.

The design strength predicted by the DSM and IS standards is unconservative. The design strength predicted by the AS/NZS 4600:2005 and Eurocode 3 (EN 1993-1-1 2005) standards is conservative. Since the predominant mode of failure of intermediate length columns is distortional, the mode of failure inferred from Eurocode 3 (EN 1993-1-1 2005) standards, AS/NZ specifications, and DSM is distortional buckling, whereas in IS method, the mode of failure inferred was flexural-torsional as there is no check for distortional buckling. Hence the IS method shows an elevated result compared to the other two specifications. So, it is crucial that the IS code be revised incorporating the distortional buckling check.

8. REGRESSION ANALYSIS

It is observed that the relationship between spacer plate slenderness ratio (λ_s), the ratio of length (a/L) and ratio of depth to width of the spacer plate (d/S) on the ultimate strength of columns under axial load varies non-linearly. From the parametric study, it is ascertained that interaction between the parameters like spacer plate slenderness ratio (λ_s), the ratio of length (a/L) and ratio of depth to width of the spacer plate (d/S) influence the strength of the columns. A non-linear regression analysis is carried out using statistical analysis software SPSS to estimate the models with arbitrary relationship between the dependent variable (σ_u/σ_y) and a set of independent variables (λ_s, a/L and d/S). This is accomplished using iterative estimation algorithms. For each iteration, parameter estimates and the residual sum of squares is obtained. For the assumed model, the sum of squares for regression, residual, uncorrected total and corrected total, parameter estimates, asymptotic standard errors, and asymptotic correlation matrix of parameter estimates are evaluated. The best fit for any assumed relationship between dependent and independent parameters can be ensured only if R-squared [1-(Residual sum of squares/Corrected sum of squares)] value is more than 0.95. The following design equation for the selected section is developed using non-linear regression analysis,

$$\sigma_u/\sigma_y=(51.972*\lambda_s)+0.009\left(\frac{a}{L}\right)+((8.261*(\lambda_s)^{0.927})*(\left(\frac{a}{L}\right)^{0.148})+(-57.550*(\lambda_s)^{0.981})*(\left(\frac{d}{S}\right)^{0.017})+0.424 \quad (1)$$

Where,

σ_u/σ_y = Ratio between the ultimate stress of the column (σ_u) and yield stress of the material (σ_y)

a/L = Ratio between the centre to centre distance between spacers(a) and the overall length (L) of the column.

d/S = Ratio between the depth of the spacers (d) and the width of the spacer (S)

λ_s = Spacer plate slenderness ratio

It is found that for the above mentioned proposed design equations (1) the R-squared value (1-(Residuals sum of squares/Corrected sum of squares)) is found to be 0.997, which is more than 0.95, and hence best fits the data obtained using nonlinear analysis.

9. DISCUSSIONS

The results presented in this paper illustrate a number of important points in relation to the performance of behaviour and ultimate capacity of an intermediate length column with spacers. From the parametric study, it is concluded that interaction between the parameters like spacer plate slenderness ratio (λ_s), the ratio of length (a/L) and ratio of depth to width of the spacer plate (d/S) influence the strength of the columns. The effect of yield stress (σ_y) and Young's modulus of elasticity (E) of the material are taken into account in the calculation of the spacer plate slenderness ratio. With the addition of spacers in the columns, the failure mode shifted from combined flexural torsional buckling and distortional mode to the interference of combined flexural buckling and distortional buckling mode. From the ultimate loads of all the columns with a number of spacers 2,3 and 4, it is inferred that the improvement in torsional rigidity increases the load carrying capacity. The percentage increase in the ultimate load of the column with different depth such as 50 mm, 75 mm and 100 mm spacers over fully opened section ranges from 6.02 to 24.5%.

10. SUMMARY AND CONCLUSIONS

An experiment on ultimate strength and buckling behaviour of intermediate length cold formed steel rack columns with and without spacers under axial compression has been conducted. The test results for open section are compared with the corresponding design strengths in Australian/ New Zealand Standards (AS/NZS 4600:2005), Direct Strength Method by North American Specifications (AISI S100-2007), Eurocode 3 (EN 1993-1-1 2005) and Indian standards (IS: 801-1975). Then the finite element model including the geometric, material non linearities and initial geometric imperfection of the specimens were developed and validated by the test results. After that, the finite element parametric analysis is conducted by varying the depth and number of spacers and the results are compared and discussed. Based on the research work above, the following conclusions can be made within the limit of the present investigation.

- Introducing the spacers has a significant effect on improvement in ultimate load capacity on the intermediate length columns.
- The ultimate load capacity increases with an increase in depth and number of spacers.
- From this investigation it is observed that the use of spacers at the proper depth and spacing helps to increase not only ultimate load capacity but also change the mode of failure.
- The provision of spacers improves the load carrying capacity by enhancing the torsional rigidity of the section.
- For open sections, the design capacity predicted by DSM and IS 801-1975 codal provisions are unconservative. But the design capacity predicted by AS/NZ, and Eurocode 3 (EN 1993-1-1 2005) codal provisions are conservative.
- Since the distortional buckling mode governs the ultimate load of the intermediate length columns, it must be taken into consideration in the design equations, which is not considered in IS 801-1975. Proper incorporation of the distortional buckling phenomena is imperative for accurate strength prediction of cold-formed steel members.
- Finally, it has been shown in this paper that finite element analysis can be used with a high level of confidence in predicting the load capacity of axially loaded cold formed steel intermediate length columns with spacers.

This investigation has also shown that further research is needed on the interaction of spacers in the ultimate load to be added in the design code provisions for intermediate length columns.

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