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New approach to improving distortional strength of intermediate length thin-walled open section columns

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ABSTRACT

This paper describes a method which can be adopted to improve the torsional and also distortional strength of thin-walled cold-formed steel columns used in pallet racking systems. Elastic buckling analysis on two different types of column sections of intermediate length was done first in our study, after finding the buckling strength and mode of failure, the column sections were made distortionally stronger by adding simple spacers. Spacers are simple concentric tubes which are used to connect the flanges of open thin walled column sections (Fig. 3.). More than 22 laboratory experiments were carried out with different spacer spacing to asses the strength and behavior of these two different column sections. All these columns tested were investigated using finite element analysis software ANSYS [1]. The experimental results were verified with finite element analysis results obtained by solving the sections using ANSYS [1] software. Details of the experimentation and finite element analysis are presented here.

KEYWORDS

Cold-formed columns, pallet racks, thin-walled sections, local buckling, distortional buckling, experimental analysis, Spacers

1. Introduction

Cold-formed steel upright section members, whose section is thin-walled, open and mono symmetric, are prone to loss of stability by interaction of two or more instability modes, such as local buckling of each component plate and overall buckling. Interaction mode between local and overall buckling is called distortional buckling. In case of the distortional buckling, each component plate distorts with lateral displacement. The occurrence of the distortional buckling depends on the sectional geometry and the length of the member. Distortional buckling occurs at intermediate length between the lengths where local and overall buckling occurs. The first part of the paper deals with the compression tests on two different types of upright sections with intermediate length. Column sections are named as Type-1 (Fig. 1.) and Type-2 (Fig. 2.). The two column sections were tested between the cross heads of a compression testing machine to find the buckling strength and mode of failure first. Later these were tested again after adding spacers at different spacing along the column. Spacers are simple concentric tubes which are used to connect the flanges of upright sections using nut and bolts as shown in Fig. 3. The difference in load carrying capacities of the two column sections with and without spacers along with the change in failure modes due to addition of spacers is presented here.

In the past, researchers have investigated the various buckling modes of commonly used cold-formed steel sections. Davies and Jiang [3] used the generalised beam theory (GBT) to analyse the individual buckling modes either separately or in selected combinations. Hancock [5] presented a detailed study of a range of buckling modes (local, distortional, and flexural–torsional) in lipped channel sections. Lau and Hancock [9] 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. Lau and Hancock [10] provided design curves for sections where the distortional buckling stress and yield stress were approximately equal.

Kwon and Hancock [6] studied simple lipped channels and a lipped channel 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 the distortional mode for the test section.

In practice, cold-formed steel thin-walled columns are manufactured with built in stiffeners to resist the local buckling to certain extent. This paper describes a way of increasing the torsional and distortional strength by using simple spacers which are added externally. The experimental results were compared with the finite element analysis results.

Finite element analysis as a tool is mainly used to verify the sections tested because, these sections being thin walled and having perforations through out length, their behaviour is quite complicated when subjected to axial loads. Shell elements available in ANSYS [1] software provide a good means to verify the experimental results.

2. Experimental study

Study-I

The experimental work was divided into two parts. In the first part the two column sections were tested to find out their buckling strength and mode of failure in a compression testing machine. The cross section of column sections is shown in Fig. 1 and Fig. 2. Effective cross section properties were found out using ANSYS [1] software after finding the b/t ratio of each segment as per BS. 5950 [2]. Type-1 column had a thickness of 1.6 mm. and Type-2 section had a thickness of 2.0 mm. The steel used in test specimens is ST52 (355MPa yield stress). Tensile coupon tests were carried out and material properties are shown in Table 1. The test set up is shown in Fig. 4. Specimens are loaded with both ends laterally restrained and vertical displacement allowed at one end in loading direction. The length of test specimens was 1100 mm for Type-1 and 1200 mm for Type-2 section. This length is chosen in the range where distortional buckling occurs in elastic buckling analysis. The steel column sections were placed between the cross-heads of a compression testing machine and loaded to failure. The ultimate loads and the nature of failure were noted. For the purpose of validation of numerical models, the strain gauges and the dial gauges were used. The strain gauges were put at mid height of upright on web portion near the slots where beam connectors are fitted. The results of the test are given in Table 3.

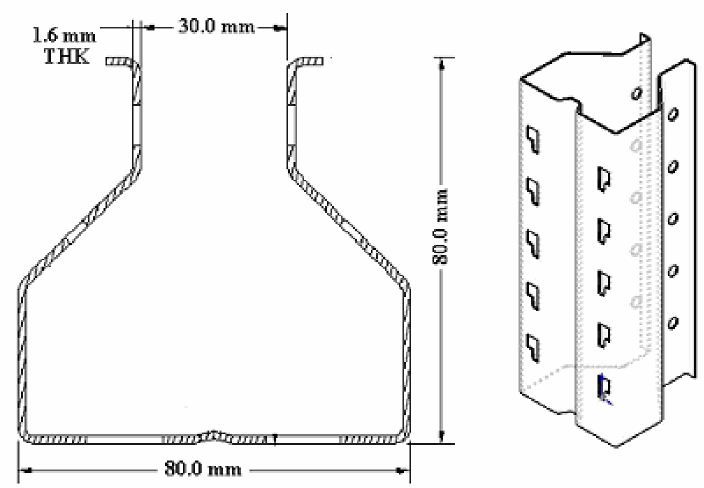


Fig. 1 Section geometries of Type -1 Section

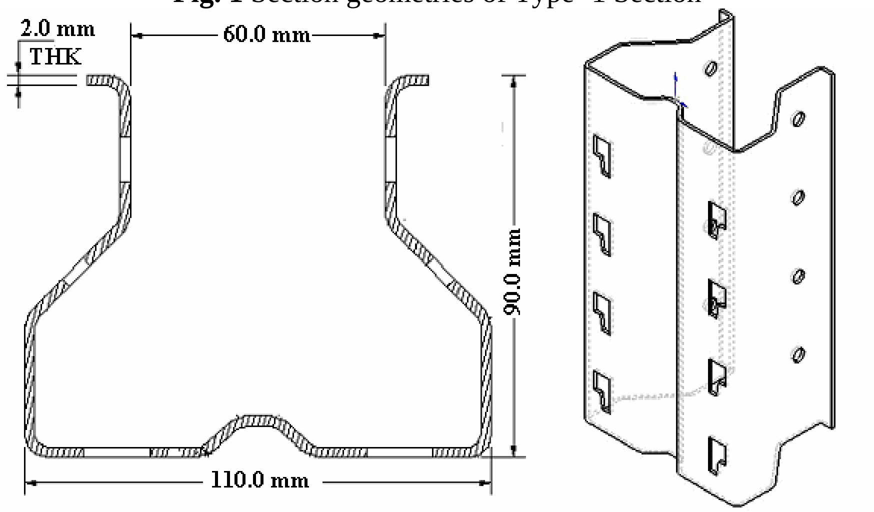


Fig. 2 Section geometries of Type-2 Section

Study-II

The two basic column sections were tested again by adding spacers. The two basic sections with spacers added are shown in Fig 3. Both the column sections were tested with varied spacing of spacers. Various spacing tried was 50, 100, 150, 200, 250, 300, 400, 500 and 550 mm. along the length of the column. These spacings were used to estimate the variation in load carrying capacity and also the change in failure mode of the basic sections. Fig. 4 shows the photograph during the test along with dial gauges. Strain gauges were also used to measure the strains for the validation of numerical models at mid height of column on web portion. The experimental and the finite element results with spacers for both the column types are compared in Table 4 and 5.

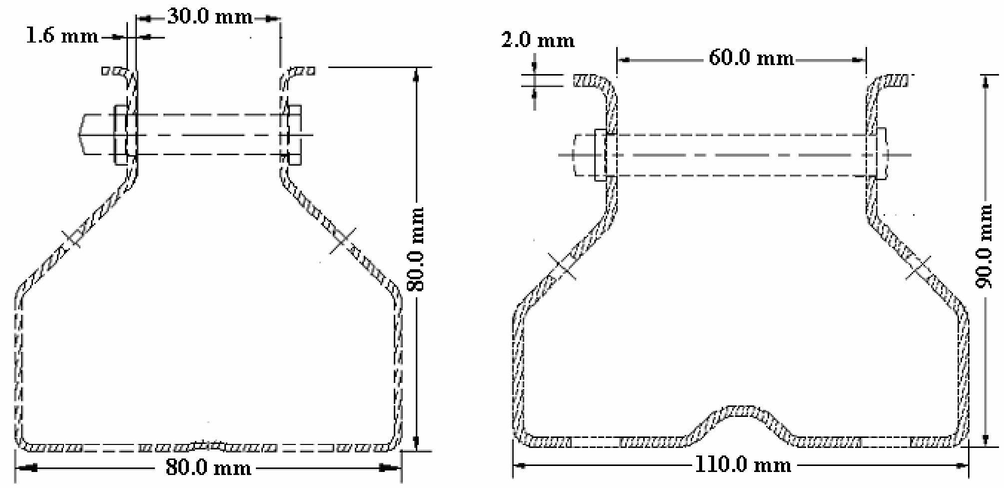


Fig. 3 Modified Type-1 and Type Column sections with spacers

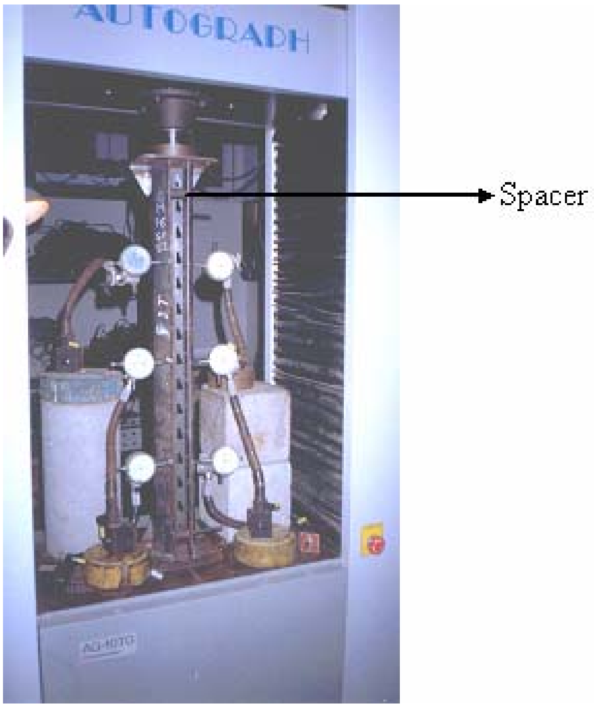


Fig. 4 Photograph of experimental set up.

3. Finite element analysis

Both linear and non-linear finite element analysis was done for all the 22 column sections tested using ANSYS [1]. Finite element analysis of Type-1 and 2 sections without spacers was carried out first and later with spacers at different spacing. A linear buckling analysis was performed to obtain the buckling loads and the associated buckling modes (see Figs. 6 and 7. and Table 2). This was followed by a non-linear ultimate strength analysis to predict the ultimate load capacity. The properties of the various finite elements used are given in Table 3. The value of the Young's Modulus was taken as 210,000 Mpa. In the later analysis, elastic-perfectly plastic material properties were assumed for both the column sections. Following a series of convergence studies, a mesh size of 5 mm × 5 mm was found to be appropriate for column length of 1200 mm. Both end conditions were laterally restrained with vertical displacement allowed in load direction and all rotations allowed. The influence of rounded corners with internal radius $r \leq 5t$ was neglected and the cross section was assumed to consist of plane elements with sharp corners.

In the non-linear analysis, initial geometric imperfections were modeled by providing initial out-of-plane deflections to the model. The first elastic buckling mode shape was used to create the geometric imperfections for the non-linear analysis. For distortional buckling modes, the imperfection was specified in terms of thickness. The maximum value of distortional imperfection was initially taken as approximately equal to the plate thickness, t, as recommended by Schafer and Pekoz [11]. Kwon and Hancock [6] found that the geometric imperfections of press-braked 1.2 mm G500 lipped channel columns were in the range of -1.5 to 1.0 mm (distortional) and -0.5 to 0.5 mm (overall). These imperfections confirm Schafer and Pekoz's [11] recommendation for distortional imperfection. Since Kwon and Hancock [6] found that the overall imperfections had little effect on the buckling of the columns of intermediate length since these columns generally buckled in a local, distortional, or mixed mode of local and distortional buckling. This study therefore did not include any overall imperfections in the FEA. Modeling imperfections for local buckling was undertaken by using imperfection values of $\frac{w}{167}$ by Schafer and Pekoz [11]. This value of imperfection is compatible with the upper limit recommended by the BS 5950 formula [2].

$$\frac{\delta_c}{t} = 0.145\left(\frac{w}{t}\right)\sqrt{\frac{f_y}{E}} \tag{1}$$

where δ_c = imperfection value t = thickness of the member w = width of the plate element of the member f_y = Yield strength of the material E = Young's modulus. For 30 mm wide plate with 1.6 mm thickness, this value is .018 mm using Eq. (1) same as $\frac{w}{167}$ is used here.

Finite element analysis of both the columns with spacers at spacing of 50, 100, 150, 200, 250, 300, 400, 500 and 550 mm was carried out. SOLID45 element is used to act as spacer for the connection between the flanges of the column sections. The same finite element properties given in Table 3 are used. The linear buckling analysis was followed by the non-linear analysis. The residual stresses were not included in the non-linear analysis since their effect on the ultimate load is considered to be negligible [11]. The test and finite element results for both the columns with spacers are compared in Tables 4 and 5.

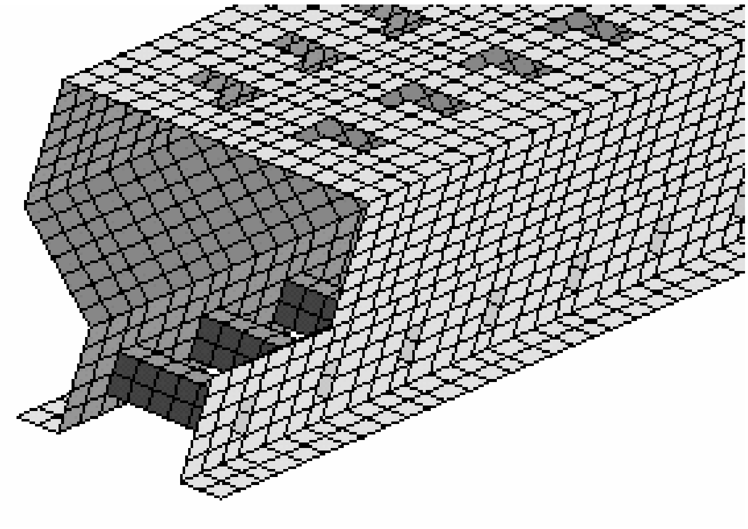
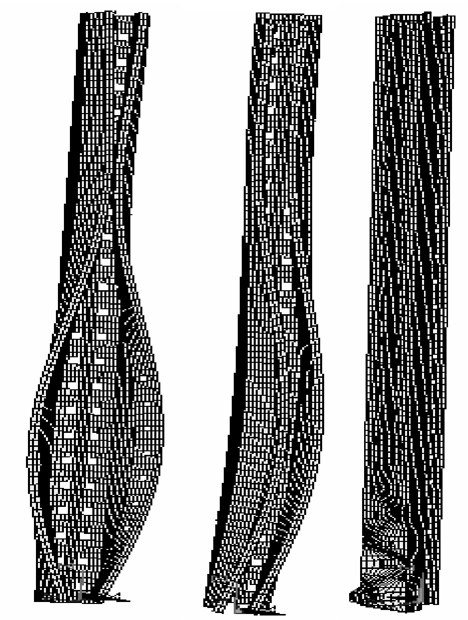


Fig. 5 Type-1 column meshing with spacers at 50 mm.



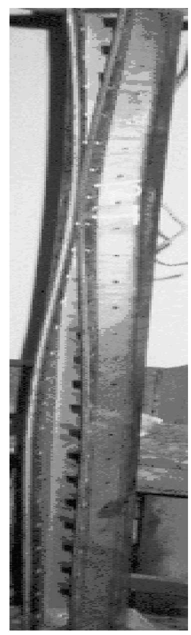
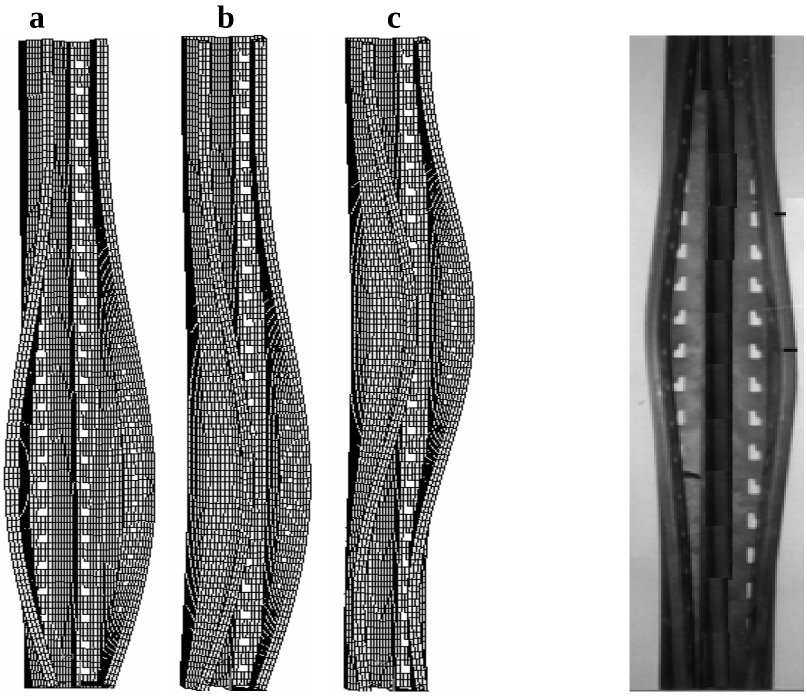


Fig. 6 Modes of Failure for Type-1 column section



Distortional Buckling | Flexural Torsional Buckling | Flexural Torsional Buckling

FEA | Test

Fig. 7 Modes of Failure for Type-2 column section

4. Comparison of experimental and FEA results

Table 2 shows that the finite element models give good predictions of the ultimate loads for the two types of columns tested in study-I. When the imperfection magnitude was assumed equal to the plate thickness, *t*, the finite element predictions were on average about 7% higher than test loads for Type-1 columns and 5% for Type-2 columns. The failure modes of both types of column models compare well with failure mode observed from experiment (see Figs. 6 and 7.)

Results obtained in Tables 4 and 5 indicate that use of spacers definitely help in increasing the ultimate load carrying capacity of both Type-1 and Type-2 sections in study-II. Finite element analysis carried out for both types of column sections give good prediction of the ultimate load carrying capacity of the columns with spacers at various intervals. FEA predictions were on average about 7% higher than the test loads for Type-1 sections as observed from Table 4 results and about 6% higher for Type-2 sections as observed from Table 5 results when imperfection magnitude was assumed equal to the plate thickness, *t*. The best spacer spacing for the Type-1 section both from the experiment and FEA analysis was found out to be 300 mm along the length and for the Type-2 sections this was 400 mm.

With the addition of spacers in both the column sections, the failure mode shifted from torsional-flexural mode to flexural mode, since the spacers help in enhancing the torsional rigidity of the sections, their by enhancing the load carrying capacity

The test and FEA strain readings are compared in Figs. 7 and 8. These readings show reasonably good agreement between the test and FEA results.

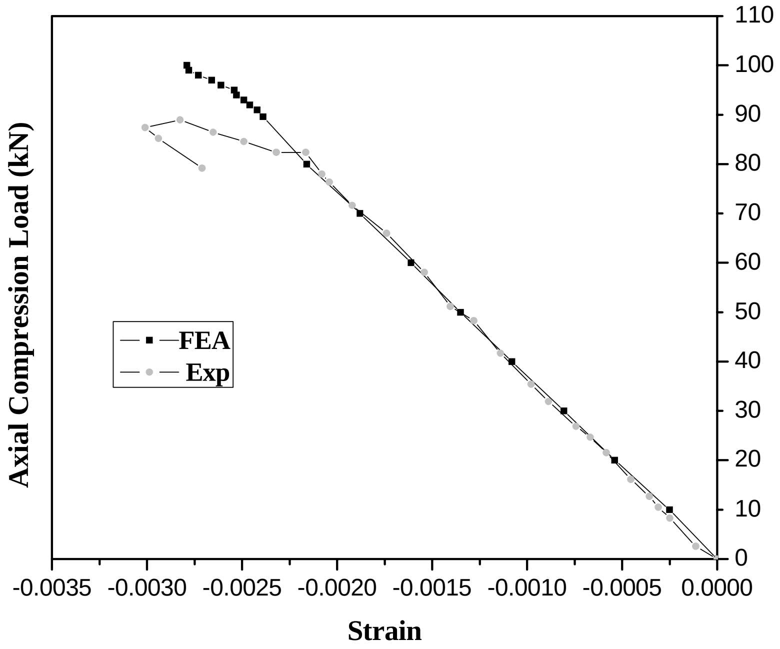


Fig. 8 Axial compression load vs Strain curve for Type-1 column (spacers at 550 mm.)

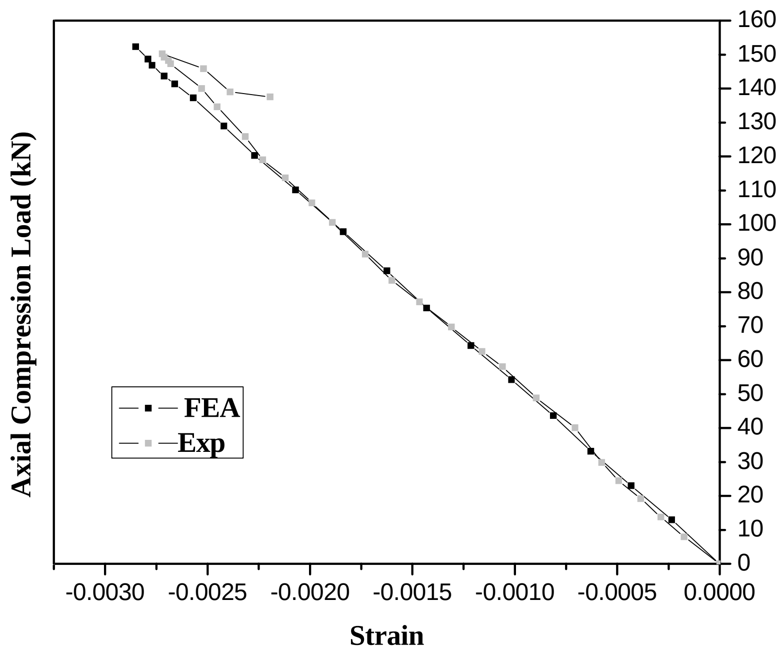


Fig. 9 Axial compression load vs. Strain curve for Type-2 column (spacers at 550 mm.)

5. Conclusions

This paper reports the results of an investigation into the use of spacers in open thin walled cold-formed steel upright sections used in pallet racking systems. Both laboratory experiments and numerical analyses were used to study their structural behaviour dominated by distortional buckling and the changes in mode of failure with use of spacers. An effect of geometrical imperfections on ultimate load was also investigated. The variations in ultimate load carrying capacities with spacers at different intervals and also without spacers were compared experimentally and by finite element analysis. This investigation has shown that use of spacers at proper intervals do help in increasing not only load carrying capacity but also vary mode of failure due to enhancement in the torsional rigidity of the sections and further research in this will help pallet racking industries at large.

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Appendix

Table 1 Material Properties

Test piece	*Average Yield Stress (2%)* σ_y (MPa)	*Average Ultimate Stress* σ_u (MPa)	*Average Young's Modulus E* (GPa)	*Elongation* (%)
Type-1	365	569	212	29
Type-2	367	572	212	30

Table 2 Comparison Buckling Load and Ultimate loads for both columns (study-I)

Column	FEA (Elastic Buckling) Load (kN)	FEA (Elastic Buckling) Failure Mode	FEA Ultimate Load (Nonlinear analysis) Zero	FEA Ultimate Load (Nonlinear analysis) *t**	Test Ultimate Load Load (kN)	Test Ultimate Load Failure Mode
Type -1	110.94	Distortional	105.09	103.99	97.37	Distortional
Type -2	161.982	Distortional	157.23	154.27	147.15	Distortional

*t** magnitude of geometric imperfection

Table 3 Properties of the Finite Elements used in the Analysis

Element name	SHELL63	SOLID45
Position of Element	Upright	Spacers
Description	Plastic shell element	3-D structural Solid Element
Number of nodes	4	8
Degrees of Freedom	*x, y, and z* translational and rotational displacements	*x, y, and z* translational displacements

Table 4 Comparison Buckling Load and Ultimate loads for Type-1 column (Study-II)

	FEA (Elastic Buckling)		FEA Ultimate Load		Test Ultimate Load	
Spacers at (mm)	Load (kN)	Failure Mode	(Nonlinear analysis) zero	*t**	Load (kN)	Failure Mode
50	157.99	Local	153.26	149.29	139.26	Local
100	147.99	Local	145.32	139.65	130.82	Local
150	145.04	Local	135.52	130.25	121.62	Local
200	143.04	Local	130.39	125.68	117.25	Local
250	142.04	Local	125.78	122.34	114.23	Local
300	141.04	Local	124.45	121.35	113.42	Local
400	141.04	Local	123.54	120.11	112.36	Local
500	140.2	Local	121.03	119.04	111.25	Local
550	139.13	Distortional	110.26	107.93	100.68	Distortional
1150	129.12	Distortional	108.28	105.32	98.25	Distortional

*t** magnitude of geometric imperfection

Table 5 Comparison Buckling Load and Ultimate loads for Type-2 column (Study-II)

	FEA (Elastic Buckling)		FEA Ultimate Load		Test Ultimate Load	
Spacers at (mm)	Load (kN)	Failure Mode	(Nonlinear analysis) zero	*t**	Load (kN)	Failure Mode
50	220.54	Flexural	233.43	223.07	210.45	Flexural
100	211.63	Flexural	224.14	213.49	201.22	Flexural
150	199..42	Flexural	210.65	201.68	190.45	Flexural
200	189.54	Flexural	196.54	191.68	180.66	Flexural
250	180.25	Flexural	186.96	181.74	171.45	Flexural
300	171.63	Flexural	182.02	178.36	168.43	Flexural
400	169.42	Flexural	173.23	170.5	160.78	Flexural
500	167.23	Flexural	170.85	165.68	156.46	Flexural
550	165.23	Flexural torsional	165.25	163.68	154.42	Flexural torsional
1150	162.39	Distortional	164.47	162.78	153.43	Distortional

*t** magnitude of geometric imperfection