INTERNATIONAL JOURNAL OF SUSTAINABLE CONSTRUCTION ENGINEERING AND TECHNOLOGY
VOL. 14 NO. 2 (2023) 114-120

IJSCET

http://publisher.uthm.edu.my/ojs/index.php/ijscet

ISSN : 2180-3242 e-ISSN : 2600-7959

International Journal of Sustainable Construction Engineering and Technology

Pre-Stress Linear and Nonlinear Buckling of Cold-Formed Steel Built-up Box Studs

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DOI: https://doi.org/10.30880/ijscet.2023.14.02.012
Received 07 February 2023; Accepted 07 February 2023; Available online 08 May 2023

Abstract: In construction industry, cold-formed steel sections become more pertinent in place of hot-rolled steel members. The design guides for CFS are inadequate and hence it is substantial to investigate their governing behaviours as material failure and structural instability (buckling). This study aims to analyse how the buckling behaviours of face-to-face built-up box short columns are administrated by the application of end plates with three different welded spacing. Six geometric models were analysed with ANSYS 2020 R1, numerical software, to evaluate the pre-stress linear and non-linear buckling loads of built-up box studs. The results distinguish the application of end plates and the most effective welded spacing for built-up geometry. It is significant that end plates are more advantageous to resist the maximum compressive loads whereas 509.6 mm is the relevant welded spacing for built-up box section. It is additionally suggested to analyse the governing of weld spacing and end conditions of various built-up geometries: box, sigma and I studs through numerical and experimental analysis.

Keywords: Pre-stress, built-up box studs, cold-formed steel, compression loads, linear and non-linear buckling

1. Introduction

Cold-formed steel (CFS) sections become more applicable in both industrial and residential buildings since 1940's due to their lightweight and thin-walled profile sections (Wei-Wen Yu and Roger A. LaBoube, 2010). Nawale et al. (2014) investigated the buckling behaviour of CFS and compared with hot rolled steel members. Material failure and structural instability called buckling are the two major categories to the sudden change in shape (deformation) of structural members. Buckling is the loss of stability of a component and is usually independent of material strength and which is one of the two limit states for compression members, columns. This loss of stability generally occurs within the elastic range of the material. End conditions of the member, eccentricity of the load, geometric imperfections and the slenderness ratio are the influential factors to buckle the compressive members. A range of their buckling modes due to load carrying capacities governs the behaviour of these thin walled members. In design evaluation stage, it is significant to eliminate or interrupt these buckling phenomena and simplifies their strength (Yerudkar at al. 2020). In terms of CFS built-up columns, significant researches were available in literature. Built-up columns connecting with two back-to-back CFS sections through batten plates, were investigated by Dabaon et al. (2015). The global buckling strength of built-up CFS channels was investigated by Fratamico et al. (2018). Without experimental results, the ANSYS finite element analysis provides estimated projections for CFS behaviours (Schafer, 2002). Krishanu et al. (2019) investigated the designed rules on the buckling behaviour of axially loaded back-to-back CFS built-up columns through experimental and FE analysis. The structural behaviour of eccentrically loaded beam-column CFS lipped and sigma section profiles were investigated numerically and experimentally by Ferhan et al. (2022). Mon and Selvam

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(2021) recommended to investigate in detail of built-up box, sigma and I sections columns under uniaxial compression loads for global buckling through the results of analytical and numerical methods. The objective of this paper is to predict how the buckling of face-to-face built-up box columns are governed by the application of end plates with three different welded spacing. To meet this purpose, pre-stress linear and nonlinear-based eigenvalue buckling of built-up specimens under uniaxial compression loads were analysed through finite element method. Numerical software of ANSYS 2020 R1 was applied in this investigation. ANSYS, one of the CAE software, is based on Finite Element Analysis which assists optimizes design assessment through their geometry, material properties, boundary conditions and load application, contact modelling and meshing. Theoretical buckling strengths of CFS members are predicted by eigenvalue problems, which must be preceded by Static Structural analysis known as pre-stress analysis that can be linear or nonlinear. Linear buckling analysis is based on eigenvalue problem and practices the perturbation method, which computes the buckling load factors and modes of deformation. Nonlinear buckling analysis accounts for material and geometric nonlinearities, load perturbations, geometric imperfections and gaps. The ultimate load for nonlinear-based eigenvalue buckling analysis is calculated by using Equation.1.

$$P_{buckling} = P_{restart} + \lambda . P_{perturbation} \qquad (1)$$

Nomenclature

$P_{buckling}$ = the ultimate buckling load of the members

$P_{restart}$ = total load in perturbation analysis at the specified restart load step

$P_{perturbation}$= perturbation load applied in buckling analysis

λ = buckling load factor for n^{th} mode

2. Numerical Investigation

2.1 Finite Element Model

To predict how the applications of end plates with different welded spacing govern on the buckling of face-to-face built-up box columns, ANSYS 2020 R1 numerical software was applied for analysing pre-stress buckling capacity of designed geometric model. For material and geometrical non-linearity, two stages of FE analysis, linear and nonlinear-based eigenvalue buckling, were performed for 10 modes of deformation. Linear-based eigenvalue analysis, firstly, was demonstrated to examine the load multipliers and modes of buckling in which the members were assumed with perfect geometry and the material as linear elastic. The lowest load factors envisaged in the first step were applied consequently to model geometric imperfections for load-displacement non-linear analysis. In the second stage, when the load applied reached a limit point sited on its equilibrium bath under the conditions of material non-linearity, geometric imperfections, the solution displayed the ultimate strength and the failure modes of buckling for cold-formed steel members.

2.2 Geometry and Material Properties

The geometric models were created through Space Claim Design Modeller with end-to-end dimensions of channel C-sections with thickness of 1.0 mm, which was comparatively smaller than other dimensions of built-up members. The section parameters of geometric models are displayed in Fig.1 and the measurements in Table 1. Due to their smaller thickness to section parameters, the conventional stress-displacement element of 4 nodes shell were used to create the built-up studs (short columns) with the height of 609.6 mm. Two symmetric sections were connected with three types of spot-welded spacing; 509.6 mm, 204.8 mm and 77.4 mm respectively. Six geometric models were created under two categories of Group A and B. The three studs in Group A were created with end plates of 100 x 100 x 6 mm, which were modelled with 8 nodes of solid elements and the rest in Group B without end plates. The yield strength and Young's Modulus were assumed as 250 MPa and 200 GPa. Poisson's ratio was assumed as 0.3.

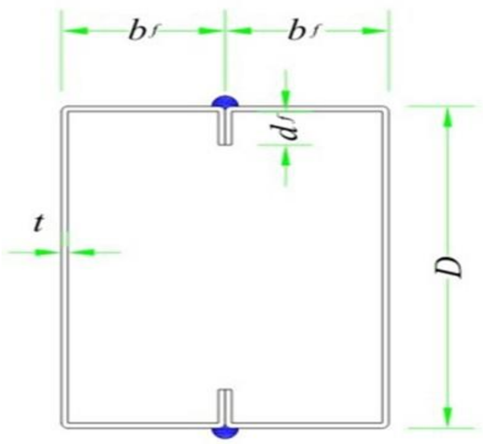


Fig. 1 - Parameters of typical tested specimens

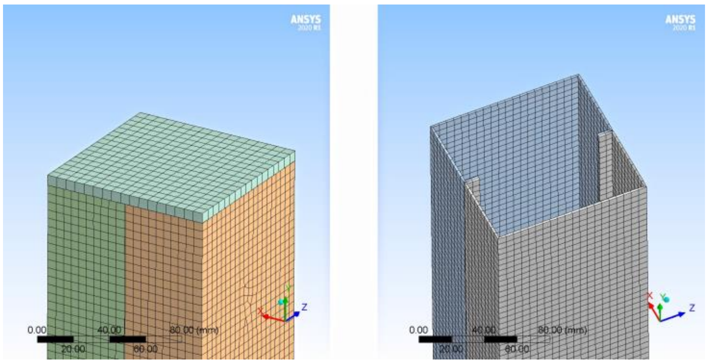
Table 1 - Parameters of tested specimens

Parameter	**Specimens**					
	A – With End plates			**B – Without End Plates**		
Thickness (t)	1.0			1.0		
Depth (D)	100			100		
Flange (b_f)	50			50		
Edge stiffener (d_f)	10			10		
Weld spacing (s)	509.6	204.8	77.4	509.6	204.8	77.4

*All of the measurements are in millimetre (mm).

2.3 Finite Element Mesh

Selection of finite element meshing prior to structural analysis is the critical step for the convergence of the model. A linear 4 nodes shell element mesh with the size of 5 x 5 mm were used whereas the end plates of 8 nodes solid models were with the size of 6 x 6 x 6 mm. Typical finite element mesh for Group A and B are illustrated in Fig. 2.

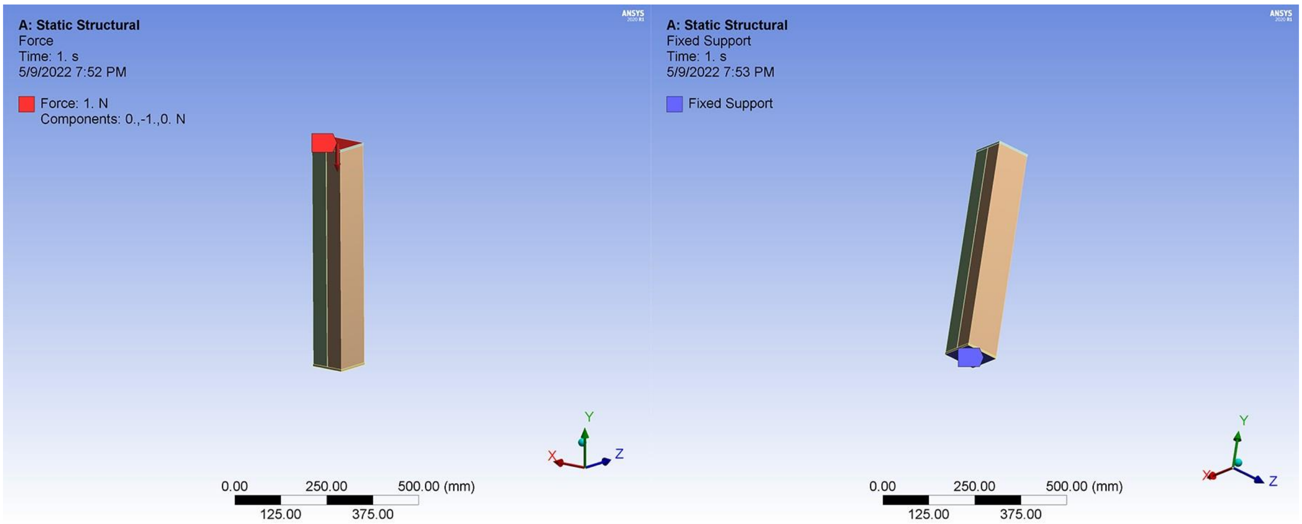


(i) Group A (ii) Group B

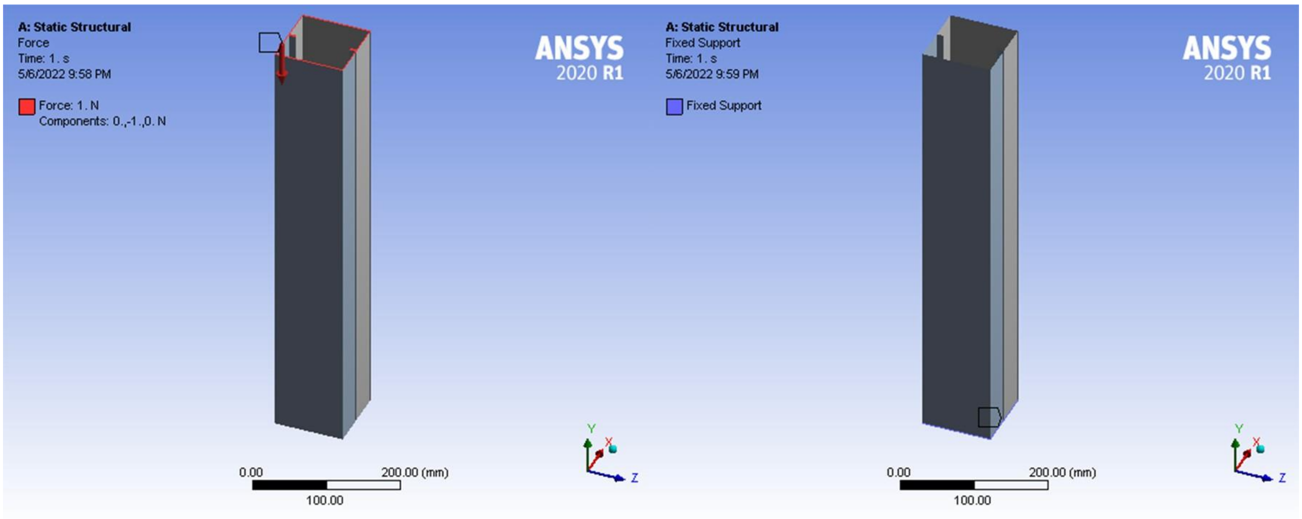
Fig. 2 - Typical finite element mesh

2.4 Boundary Conditions and Load Application

The centroids of the built-up columns were assumed as the centre of gravity for axial compression loads. The reaction ends of the columns were modelled as fixed end and the load end as the free one. The translation and rotation at the bottom ends of the columns were restrained in all directions. The loads were applied at the centre of the upper free ends along the negative Y direction. The typical boundary conditions and load application are illustrated in Fig.3 (i) & (ii).



(i) Load application and boundary conditions for Group A



(ii) Load application and boundary conditions for Group B

Fig. 3 - Typical boundary conditions and load application

2.5 Contact Modelling

"Surface to surface" contact was applied for the interaction between the cross sectional edges of the columns and solid end plates of the geometric models in Group A. The edges of the cross section at the both ends performed as the contact bodies and the inner surfaces of the end plates as the target ones. MPC formulation is used as bonded contact. There were no penetrations between the contact surfaces and these were applied only for the models in Group A.

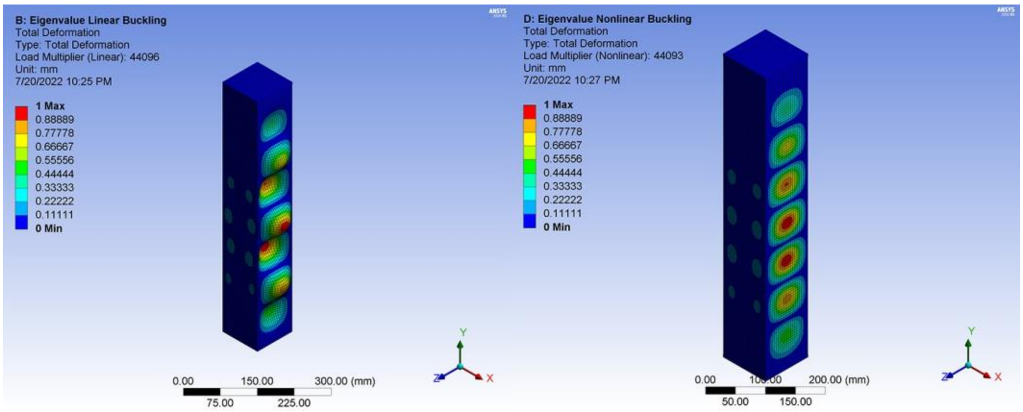
3. Result and Discussion

Table 2 displays the linear and non-linear buckling load of Group A & B in 10 modes of deformation. Fig.4 (i) & (ii) compare the pre-stress linear and non-linear buckling of AS1 and BS2.

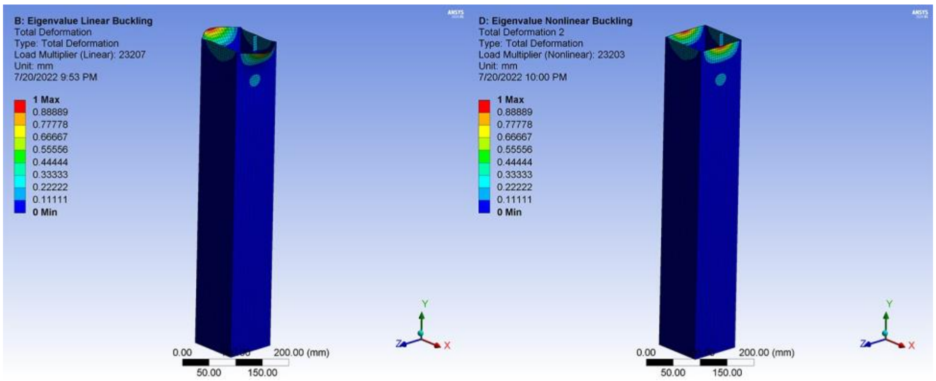
Table 2 - Linear and non linear buckling load of group A & B

Specimens	Modes	Linear Buckling Load (kN)	Non-Linear Buckling Load (kN)
	1	44.096	44.094
	2	44.14	44.138
	3	44.847	44.84
	4	44.934	44.927

AS-1	**5**	**46.117**	**46.115**
	6	**46.393**	**46.391**
	7	**46.739**	**46.733**
	8	**47.261**	**47.254**
	9	**49.646**	**49.645**
	10	**50.294**	**50.289**
AS-2	**1**	**44.096**	**44.094**
	2	**44.14**	**44.138**
	3	**44.847**	**44.84**
	4	**44.934**	**44.927**
	5	**46.117**	**46.115**
	6	**46.393**	**46.391**
	7	**46.739**	**46.733**
	8	**47.262**	**47.254**
	9	**49.646**	**49.645**
	10	**50.294**	**50.289**
AS-3	**1**	**44.096**	**44.094**
	2	**44.14**	**44.138**
	3	**44.847**	**44.84**
	4	**44.934**	**44.927**
	5	**46.117**	**46.115**
	6	**46.393**	**46.391**
	7	**46.739**	**46.733**
	8	**47.262**	**47.254**
	9	**49.646**	**49.645**
	10	**50.294**	**50.289**
BS-1	**1**	**22.162**	**22.16**
	2	**22.403**	**22.403**
	3	**43.442**	**43.437**
	4	**43.926**	**43.923**
	5	**44.159**	**44.148**
	6	**44.666**	**44.658**
	7	**45.196**	**45.191**
	8	**45.922**	**45.912**
	9	**46.316**	**46.312**
	10	**46.898**	**46.889**
BS-2	**1**	**23.207**	**23.204**
	2	**23.482**	**23.482**
	3	**43.851**	**43.849**
	4	**44.155**	**44.153**
	5	**44.599**	**44.591**
	6	**44.918**	**44.911**
	7	**45.617**	**45.613**
	8	**46.349**	**46.339**
	9	**46.673**	**46.67**
	10	**47.267**	**47.26**
BS-3	**1**	**22.163**	**22.16**
	2	**22.404**	**22.403**
	3	**43.443**	**43.437**
	4	**43.927**	**43.923**
	5	**44.16**	**44.148**
	6	**44.667**	**44.658**
	7	**45.197**	**45.191**
	8	**45.923**	**45.912**
	9	**46.317**	**46.312**
	10	**46.899**	**46.889**



(i) Pre-stress linear and non linear buckling of AS1



(ii) Pre-stress linear and non linear buckling of BS2

Fig. 4 - Comparison of linear and non linear buckling of AS1 & BS2

In Group A, the results indicate there is no much difference among the built-up specimens. Eigenvalue linear and non-linear buckling of specimens with different welded spacing in all modes display the same value, the minimum 44.096 kN and maximum 50.294 kN for linear and 44.094 kN and 50.289 kN for non-linear. This grants there is no influence of welded spacing for the specimens in Group A. The results for Group B specimens, however, are not exactly the same and vary justified on their welded spacing. Among three specimens, BS1, BS2 and BS3, the linear and non-linear buckling of BS2 display the highest value, 23.207 kN and 47.267 kN. These data indicate that the welded spacing of 204.8 mm governs the maximum buckling loads in Group B. Fig. 5 illustrates the comparison of these numerical results.

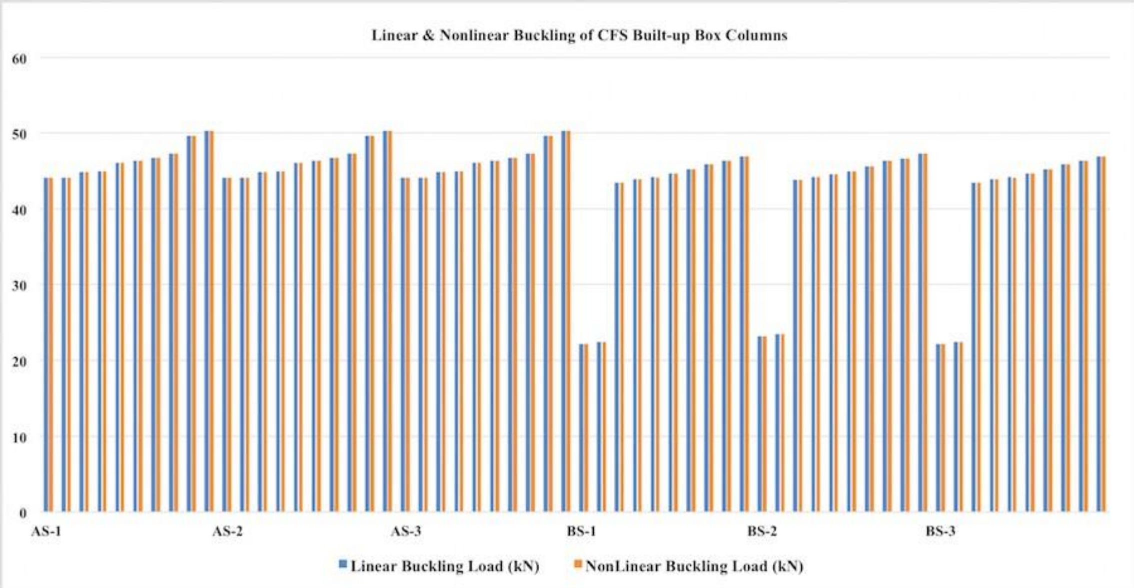


Fig. 5 - Comparison of linear and non linear buckling of Group A & B

4. Conclusions

For face-to-face built-up box section, the results reveal that the specimens with end plates are more applicable rather than without end plates. The results of the first two modes of deformation, the maximum compressive loads, in the former are nearly double than those in the latter though there are slightly the same load in the rest modes. The studs, consequently, with end plates are more pertinent for uniaxial compressive loads. For Group A specimens, there is no much variances between their welded conditions and henceforth 509.6 mm is reasonable to create built-up box studs. It is recommended to associate the influence of welded spacing and end conditions on buckling loads among the various built-up studs: box, sigma and I sections through numerical and experimental analysis.

Acknowledgement

The authors would like to thanks the Department of Civil Engineering, Faculty of Engineering, Lincoln University College for allowing to conduct this research.

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