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Buckling of Cold-Formed Steel Built-up I Studs

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ABSTRACT: Cold-formed steels become comparatively introduced in place of conventional steels due to their lightweight, high strength to weight ratio and being environmental friendly materials. Structural instability (buckling), web crippling and low ductility are the major concern in their application. This paper focuses on buckling behaviours of back-to-back cold-formed steel built-up I studs through numerical investigation. Two groups of built-up I studs with and without end plates with three different weld spacing were analysed with ANSYS 2020 R1. The results reveal that local buckling governs for the studs with end plates and local-distortional for without end plates.

KEYWORDS: Cold-formed Steel, Back-to-back I Studs, ANSYS 2020 R1, Local Buckling, Local-distortional Buckling

I. INTRODUCTION

Cold-formed steel (CFS) doesn't require heat too much to form their shapes and is manufactured at room temperature unlike hot-rolled steel (HRS) and produced by means of roll forming, folding and press braking [1]. With various thickness of steel sheet from 0.5mm to 3mm, cold-formed steel channel sections (CFSCS) are being widely applied as popularized materials for roof truss system in place of traditional timber structure [2]. CFS are not able to conquer their full strength due to the geometric shapes of cross sections and length of each profile depending on the various modes of buckling behaviours. Due to imperfection and insufficient of design theory for complex nature of interactive buckling modes, local-distortional (LD) and local-distortional-global (LDG), of CFSS, it becomes more reliable on experimental test data and results. With the advancement of numerical methods such as the finite element method (FEM) at present, true ultimate strength solutions including all-important checks and failure modes can be more accessible than experimental investigation.

Fratamico et al (2018) studied the effectiveness of the use of EFGs to the built-up I columns [3].James B.P. Lim (2019) developed the numerical models of back-to-back light gauge CFS built-up stud and slender columns under compression with the use of ABAQUS and likened with the experimental results of their axial strength [4]. Krishanu Roy (2019) proposed the novel design rules with the application of numerical software; ANSYS and ABAQUS, on the deformed shapes and buckling modes of axially compressed back-to-back CFS built-up columns [5]. Yao (2021) analysed the slenderness ratio, the spacing of screws, and the end fastener group of built-up I section columns with the aid of ABAQUS software and compared the outcomes with the experimental results [6]. Muthuraman Mohan et al (2022) studied elastic and non-elastic buckling behaviour of web-stiffened cold-formed steel back-to-back built-up columns by means of numerical software ABAQUS [7]. The approaches for the models of 36 built-up columns were incorporated as reported by Anbarasu et al [8]. Mon and Selvam investigated that 509.6 mm is the best weld spacing for CFS with fillet welded connections through experimental approach [9].

The objective of this analysis is to predict how the buckling of back-to-back built-up Istuds are governed by the application of with and without end plates. 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.

$P_{buckling}=P_{restart}+\lambda.P_{perturbation}$Equation 1

Where,

$P_{buckling}$ = 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

II. NUMERICAL METHOD

To predict how the applications of with and without end plates with three different fillet-welded spacing (509.6 mm, 204.8 mm, 77.4 mm) govern on the buckling of back-to-back built-up Istuds, 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.

III. GEOMETRY AND MATERIAL PROPERTIES

The geometric models of built-up I studs were crafted through ANSYS 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 model is displayed in (Fig 1) and the measurements in Table 1. Due to the 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 spot-welded spacing; 509.6 mm, 204.8mm & 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.

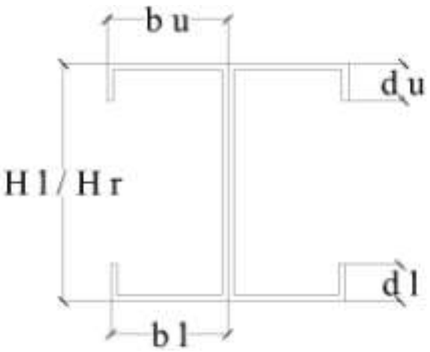
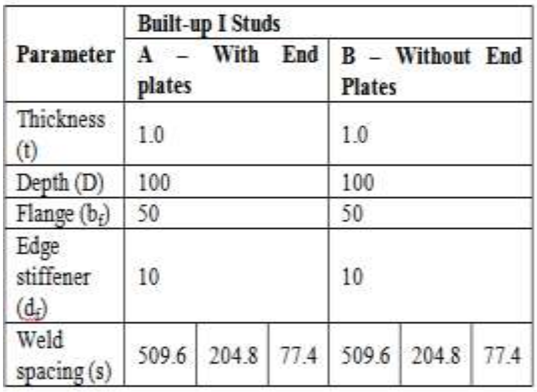


Figure 1: Typical Dimension of Specimens in ANSYS



Parameter	Built-up I Studs					
	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

Table 1: Parameters of Tested Specimens A & B

IV. 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.

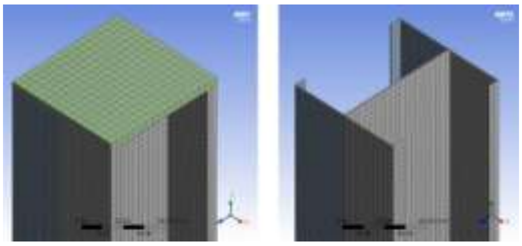

(i) Group A (ii) Group B

Figure 2: Typical Finite Element Mesh

V. BOUNDARY CONDITION & 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.

VI. 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 Groups 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 Groups A.

VII. RESULTS & DISCUSSION

Table 2 displays the linear and non-linear buckling load of Group A & B in 10 modes of deformation. The pre-stress linear and non-linear buckling of built-up I studs are compared in (Fig. 3 (i) to (vi)).

Table 2: Linear and Non Linear Buckling Loads & Modes of Group A & B

Specimens	**Modes**	**Linear Buckling Load (kN)**	**Mode of Buckling**	**Non-Linear Buckling Load (kN)**	**Mode of Buckling**
AS-1	1	40.374	Local Buckling	39.717	Local Buckling
	2	41.192	Local Buckling	40.123	Local Buckling
	3	43.519	Local Buckling	41.967	Local Buckling
	4	44.452	Local Buckling	42.354	Local Buckling
	5	46.104	Local Buckling	45.055	Local Buckling
	6	47.758	Local Buckling	46.060	Local Buckling
	7	51.401	Local Buckling	49.943	Local Buckling
	8	56.752	Local Buckling	53.652	Local Buckling
	9	60.751	Local-Distortional Buckling	57.464	Local-Distortional Buckling
	10	62.519	Local Buckling	59.227	Local Buckling
AS-2	1	40.260	Local Buckling	39.166	Local Buckling
	2	40.689	Local Buckling	39.775	Local Buckling
	3	42.118	Local Buckling	40.715	Local Buckling
	4	44.161	Local Buckling	42.109	Local Buckling
	5	45.432	Local Buckling	43.86	Local Buckling
	6	46.484	Local Buckling	45.334	Local Buckling
	7	49.534	Local Buckling	47.795	Local Buckling
	8	55.171	Local Buckling	52.225	Local Buckling
	9	57.577	Local-Distortional Buckling	55.631	Local-Distortional Buckling
	10	60.443	Local-Distortional Buckling	57.559	Local-Distortional Buckling
	1	40.277	Local Buckling	39.712	Local Buckling

AS-3	2	40.945	Local Buckling	40.114	Local Buckling
	3	43.359	Local Buckling	41.964	Local Buckling
	4	44.419	Local Buckling	42.339	Local Buckling
	5	46.049	Local-Distortional Buckling	45.052	Local Buckling
	6	47.604	Local Buckling	46.053	Local Buckling
	7	51.344	Local Buckling	49.939	Local Buckling
	8	56.633	Local Buckling	53.636	Local Buckling
	9	60.432	Local Buckling	57.447	Local Buckling
	10	62.494	Local-Distortional Buckling	59.231	Local-Distortional Buckling
BS-1	1	22.645	Distortional Buckling	20.992	Distortional Buckling
	2	40.718	Distortional Buckling	39.919	Distortional Buckling
	3	40.860	Local Buckling	40.219	Local Buckling
	4	43.109	Local-Distortional Buckling	41.376	Local-Distortional Buckling
	5	43.383	Local-Distortional Buckling	42.608	Local-Distortional Buckling
	6	45.188	Local-Distortional Buckling	43.685	Local-Distortional Buckling
	7	47.202	Local-Distortional Buckling	46.205	Local-Distortional Buckling
	8	49.535	Local-Distortional Buckling	48.623	Local-Distortional Buckling
	9	53.331	Local-Distortional Buckling	52.024	Local-Distortional Buckling
	10	54.964	Local-Distortional Buckling	54.077	Local-Distortional Buckling
BS-2	1	22.645	Local-Distortional Buckling	20.992	Local-Distortional Buckling
	2	40.718	Local-Distortional Buckling	39.919	Local-Distortional Buckling
	3	40.860	Local Buckling	40.219	Local Buckling
	4	43.109	Local-Distortional Buckling	41.376	Local-Distortional Buckling
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BS-3	1	22.645	Local-Distortional Buckling	20.992	Local-Distortional Buckling
	2	40.718	Local-Distortional Buckling	39.919	Local-Distortional Buckling

	3	40.86	Local Buckling	40.219	Local Buckling
	4	43.109	Local-Distortional Buckling	41.376	Local-Distortional Buckling
	5	43.383	Local-Distortional Buckling	42.608	Local-Distortional Buckling
	6	45.188	Local-Distortional Buckling	43.685	Local-Distortional Buckling
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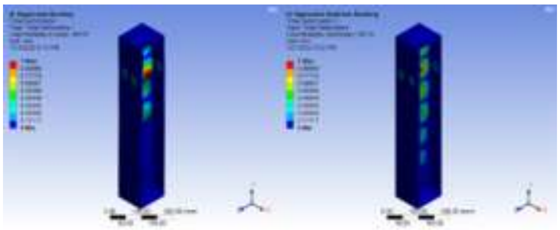
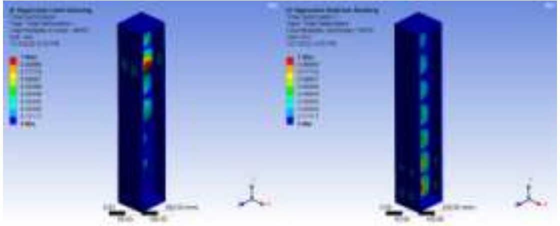
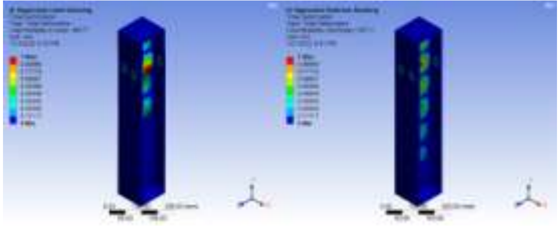


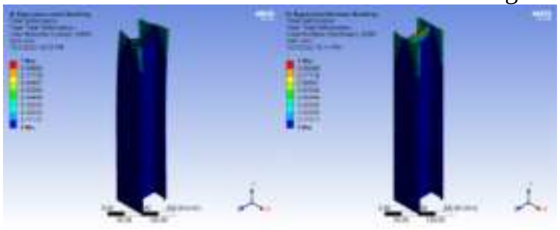
Pre-Stress Linear and Non Linear Buckling of AS1



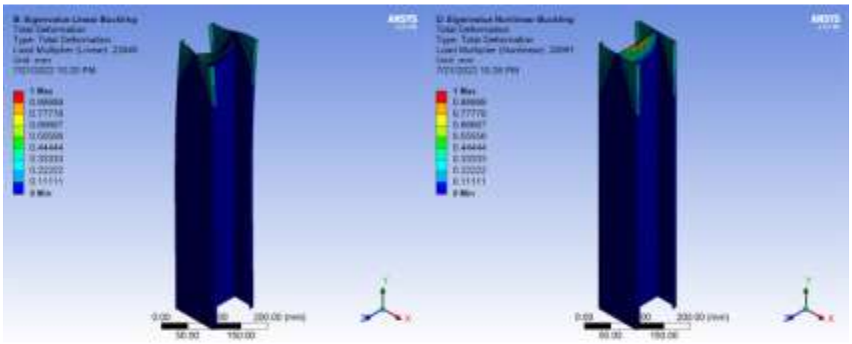
(i) Pre-Stress Linear and Non Linear Buckling of AS2



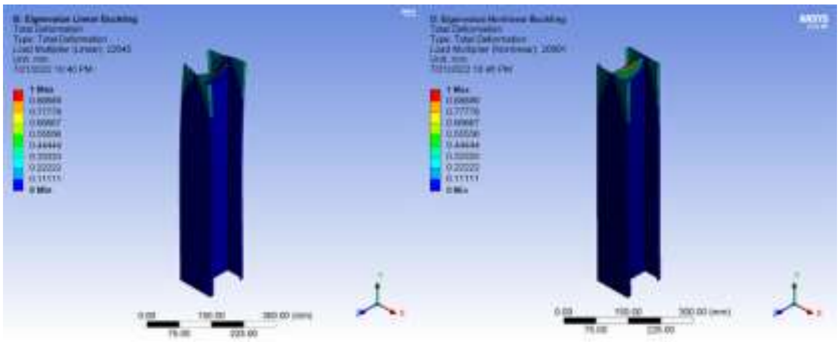
(ii) Pre-Stress Linear and Non Linear Buckling of AS3



(iii) Pre-Stress Linear and Non Linear Buckling of BS1



(iv) Pre-Stress Linear and Non Linear Buckling of BS2



(v) Pre-Stress Linear and Non Linear Buckling of BS3

Figure 3: Comparison of Linear and Non Linear Buckling of Group A&B

Among the Group A studs, AS1 displays the maximum load in both linear and nonlinear analysis nevertheless the load in Group B studs exhibits the same in all types welded spacing. The results summarize that the welded spacing do not have excessive influence on the failure loads and it is endorsed that 509.6 mm is applicable spacing for built-up columns. As the end conditions of studs, the load bearing capacity of studs with end plates are greater than those without end plates. In Group A, built-up I studs with endplates, local buckling governs in all modes except in 9th mode of AS1 where local-distortional buckling occurs. For AS2 and AS3, local-distortional occurs at the last two modes while the remaining modes are governed by local buckling. Local-distortional buckling mainly appears in all modes of Group B members, built-up I studs without endplates. Comparing the results identify that major failure mode is local especially for built-up I studs with endplates whereas local-distortional is the key failure mode for built-up I studs without endplates. Numerical analysis proves that global buckling does not take place in all modes of built-up I studs either with or without endplates. The numerical outputs endorsed that 509.6 mm is the appropriate weld spacing for fillet welded connections.

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