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Investigation on the Strength and Behavior of Cold-Formed Steel Angle Columns

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Abstract: This article investigates the numerical and theoretical study on the buckling behaviour of cold formed steel lipped angle columns under pinned end conditions. The sections were analyzed using ABAQUS software. Geometric and material non linearities were included in the model. Parametric study was conducted by varying the thickness and length of the specimens. Three types of sections were chosen for this study based on the geometric limitations for the prequalified sections provided in the North American Specifications for Cold formed steel structures (AISI S100 – 2007). The Analysis was conducted on 24 specimens. All the specimens were failed under the combination of Local and Distortional Buckling. Theoretical study was carried out using Direct Strength Method as per North American Specifications for Cold formed steel structures. The Numerical results were compared with the Direct Strength method. Based on the comparison of results suitable recommendations were suggested in the direct strength method.
Keywords: Cold formed Steel, Finite Element method, Direct Strength method.

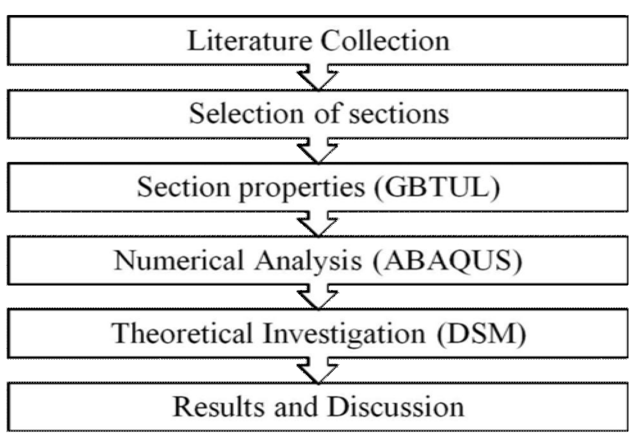
I. INTRODUCTION

Cold formed steel sections are thin sections made out of thin sheets of steel by rolling or press braking method in cold state. These sections are having uniform thickness. These sections are also called Light Gauge Steel Sections or Cold Rolled Steel Sections. Cold formed steel is used as secondary structural members like purlins and girts. Cold formed steel sections are thin in cross section and fails by buckling prior to yielding. Different modes of failure are observed in cold formed steel like local buckling, distortional buckling and lateral distortional buckling.

II. OBJECTIVE OF STUDY

1) To examine the buckling behaviour and load carrying capacity of cold formed steel plain and lipped angle columns under axial load.
2) Three types of angle sections were chosen by varying thickness and length.
3) The material properties of the angle section were found by GBTUL.
4) The ultimate load carrying capacity is to be compared with theoretical investigation done using Direct Strength method.

III.METHODOLOGY

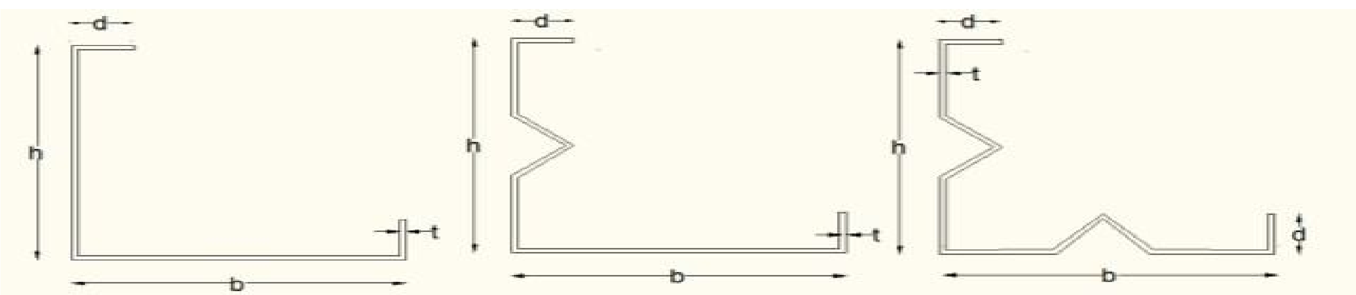


IV. LITERATURE REVIEW

=B. W. Schafer (2002) reports that open cross-section, thin-walled, cold-formed steel columns have at least three competing buckling modes: local, distortional, and Euler i.e., flexural or flexural- torsional buckling. Ben young and Jintang Yan (2002) presented the design and numerical investigations into the strength and behaviour of cold-formed lipped channel columns using finite element analysis. A non-linear finite element model is developed and verified against fixed ended channel column tests. S. Narayanan, M.Mahendran (2003) studied the distortional buckling behaviour of a series of innovative cold-formed steel columns. More than 15 laboratory experiments were undertaken first on these innovative steel columns of intermediate length under axial compression. Ben Young and Ehab Ellobody (2005) described the buckling behaviour of cold formed steel equally lipped angle columns. The initial local imperfections, residual stresses, and corner material properties of the cold-formed steel angles have been measured experimentally. Ben Young, Ehab Ellobody (2007) reported the design of cold-formed steel unequal angle compression members have been investigated. A finite element model (ABAQUS) for the analysis of cold-formed steel lipped angle sections with unequal flange widths has been presented.

M.Anbarasu and Dr.S.Sukumar (2010) studied the buckling behaviour of open web Open cross section with intermediate stiffener & corner Lips under compression. Expressions for distortional buckling stress & flexural torsional buckling stress has been obtained for mono symmetric open cross section compression members. Pedro B. Dinis, Dinar Camotim (2010) reported the results of a numerical investigation concerning the elastic and elastic–plastic post buckling behaviour of cold-formed steel lipped channel columns affected by distortional/global (flexural–torsional) buckling mode interaction. Young & Hancock (2012) experimentally investigated the cold formed steel channel section to combined bending and web crippling. A series of tests on unlipped channels rolled from high strength steel with thickness of 6mm and maximum web slenderness of 45 were conducted. Eliane S.dos Santos, Eduardo M.Batista, Dinar Camotim (2012) presented the structural behaviour and ultimate strength of fixed-ended cold-formed steel lipped channel columns experiencing local–distortional–global buckling mode interaction. Y.Shifferaw, B.W.Schafer (2014) presented the significant post buckling reserving local buckling that has been observed intersection cold-formed steel angle columns, and to provide design guidance for locally slender cold-formed steel lipped and plain angle columns with fixed end boundary conditions.

V. CROSS SECTION OF THE SELECTED SPECIMEN



TYPE 1 TYPE 2 TYPE 3

Fig 1.Cross section of selected sections

A. *Material Properties*

Yield stress (fy) = 250 N/mm^2

Modulus of Elasticity = 2x10^5 N/mm^2

Poisson's ratio = 0.3

Table 1 Dimensions of selected sections

Section ID	Depth (h) (mm)	Width (b) (mm)	Depthof lip (d) (mm)	Thickness (t) (mm)	Length (L) (mm)	Inclination (θ)
Type 1	80,90	80,90	15	1.6,2	1000, 1200	0
Type 2	80,90	80,90	15	1.6,2	1000, 1200	45
Type 3	80,90	80,90	15	1.6,2	1000,1200	45

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VI. MODELLING OF PROPERTIES

The cross sectional dimensions of the selected section is inputted in the PART section along with the length and the material properties of the section such as the youngs modulus, poison ratio, yield stress, etc...Since ABAQUS does not have any predefined input unit, we have to uses throughout the analysis. The reference points are created in the centroid of the section at top and bottom of the sections and the top and bottom nodes are separately connected with their respective "Multi Point Constraint (MPC)" with the connecting elements as "beam element".

Fig 2 Creation of Mesh

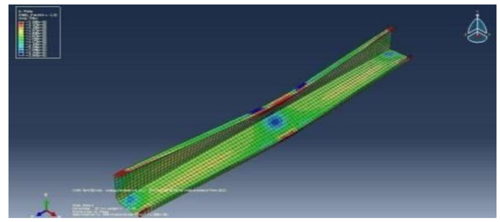

Fig 3 Deformed shape of the selected section

VII. DIRECT STRENGTH METHOD

The direct strength in the Australian/New Zealand Standard (AS/NZS 2005) for cold cold formed steel structures was adopted from the North American Specification (NAS 2004) for the design of cold formed steel structural members. The design rules of the direct strength method in the AS/NZS Standard are identical to those in the NAS Specification. The nominal axial strength or un factor design strength (P_{DSM}) is the minimum of the nominal axial strengths for flexural buckling (P_{nc}), local buckling (P_{nl}), and distortional buckling (P_{nd}).

1) *Section Properties*

Gross area of the section $A = 336\ mm^2$

$E = 2.00E+05\ N/mm^2$

$G = 76923.08\ N/mm^2$

Moment of inertia about XX axis $I_{xx} = 6.91\ E+05\ mm^4$

Moment of inertia about YY axis $I_{yy} = 1.93\ E+05\ mm^4$

$r_O = 58.72\ mm$

$J = 5.60E+02\ mm^4$

Yield stress of material $fy = 250\ N/mm^2$

Warping constant $I_w = 2.92E+07\ mm^6$

Radius of gyration about XX axis, $r_x = 45.34\ mm$

Radius of gyration about XX axis, $r_y = 23.97\ mm$

$X_{cg} = 26.250mm$

2) *Local Buckling*

Factor = 1.867 from buckling plot

Critical elastic local column buckling load,Pcrl = 156.800 KN

$$Pnl = \left[1 - 0.15\left(\frac{P_{crl}}{P_{ne}}\right)^{0.4}\right]\left(\frac{P_{crl}}{P_{ne}}\right)^{0.4} P_{ne}$$

Where $\lambda l = \sqrt{\ \ \ / P_{crl}} = 0.776$

Nominal strength for local buckling = 45.828 Kn

3) *Distottional Buckling*

Factor = 0.6307 (from buckling plot)Critical elastic buckling Load ,

Pcrd = 33.90 kNFor λd≤0.56188

Pnd = Py

For λd≤0.561

Nominal axial strength distortional buckling,

Pnd = 32.106 kN

Nominal compressive strength,

Pn = 51.627 kN

Pnd = 45.828 kN Nominal compressive strength

VIII. RESULTS AND DISCUSSION

The results obtained from the Numerical and Theoretical study are listed in the table

Section type	Thickness (mm)	Length (mm)	Yield stress (N/mm^2)	PFEM (kN)	PDSM (kN)	P_{FEM} / P_{DSM}	P_{DSM} / P_{FEM}
TYPE 1 80X80	1.6	1000	250	25.35	47.13	0.54	1.86
		1200		20.07	39.52	0.51	1.97
	2	1000		34.16	60.79	0.56	1.78
		1200		29.55	54.49	0.54	1.84
TYPE 1 90X90	1.6	1000	250	27.09	50.50	0.54	1.86
		1200		21.07	41.70	0.51	1.98
	2	1000		34.94	65.05	0.54	1.86
		1200		25.4	45.83	0.55	1.80
TYPE 2 80X80	1.6	1000	250	35.76	67.53	0.53	1.89
		1200		32.07	57.15	0.56	1.78
	2	1000		37.16	72.07	0.52	1.94
		1200		37.95	75.28	0.50	1.98
TYPE 2 90X90	1.6	1000	250	39.07	59.25	0.66	1.52
		1200		46.94	92.82	0.51	1.98
	2	1000		40.47	77.96	0.52	1.93
		1200		32.23	61.79	0.52	1.92
TYPE 3 80X80	1.6	1000	250	32.23	61.79	0.52	1.92
		1200		47.24	88.58	0.53	1.88
	2	1000		34.26	63.54	0.54	1.85
		1200		45.17	83.06	0.54	1.84
TYPE 3 90X90	1.6	1000	250	44.34	81.34	0.55	1.83
		1200		39.78	72.63	0.55	1.83
	2	1000		55.93	103.95	0.54	1.86
		1200		49.55	93.62	0.53	1.89

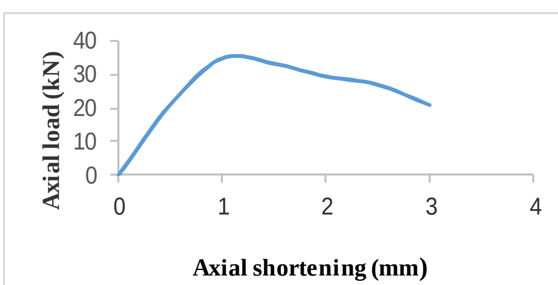

Fig 4 Axial Load vs Displacement for Type 1 section

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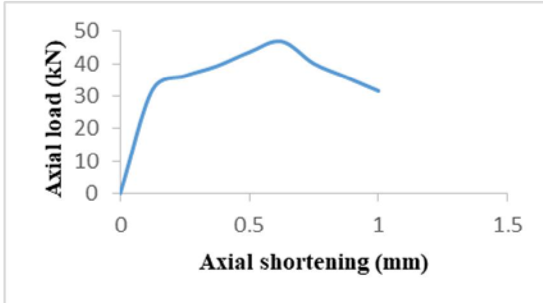

Fig 5 Axial Load vs Displacement for Type 2 section

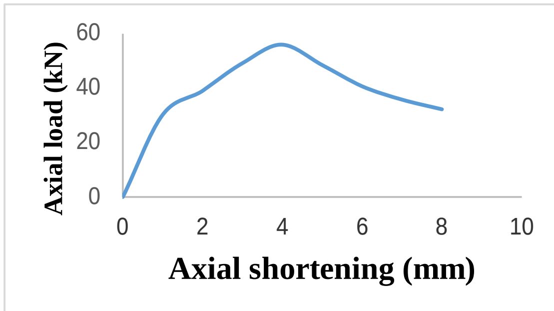

Fig 6 Axial Load vs Displacement for Type 3 section

A. Proposed Equation

The curve shown in Fig 5.1 is drawn by plotting DSM values in Y axis and our obtained FEM values in X axis. The values predicted by DSM are slightly higher than FEM, which seems to be over predict the member capacity. Hence the new design expression is proposedas below,

$$P_{FEM} = 1.8585P_{DSM} + 0.0912$$

After finding the DSM values for the sections the corresponding FEM values are also determined from the equation proposed. Based on this equation we can find the ultimate load carrying capacity of the columns under the distortional buckling.

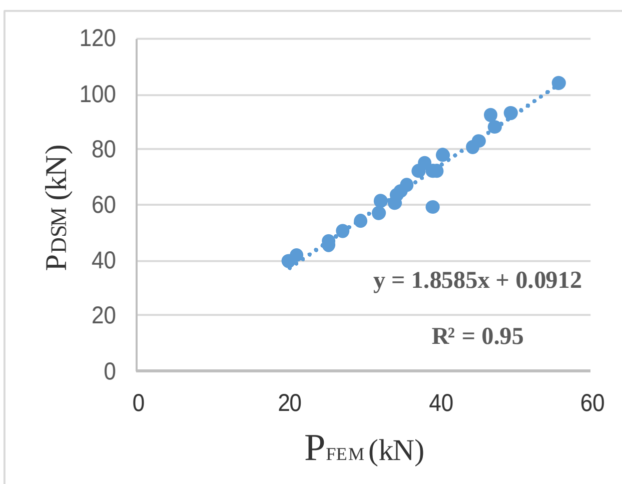

Fig 7 FEM vs. DSM Curve

IX. CONCLUSIONS

A series of angle sections has been analyzed to investigate the interaction between local and distortional buckling and its ultimate strength. Numerical and theoretical studies were done. Section properties were found using GBTUL. Numerical analysis was carried out using ABAQUS and Theoretical study was carried out using Direct Strength Method. Material properties and geometric imperfection wereincluded.

The section profiles were chosen based on the geometric limitation for the prequalified sections provided in the IS 801 draft code. A series of parametric studies was also carried out by varying thickness (1.2, 1.6, 2 mm) and length (1000, 1200 mm) with yield strength of 250MPa. The column strengths obtained from the FEM are compared with the design column strengths calculated using the Direct StrengthMethod.

Based on the investigation made on the hinged ended cold formed steel angle column members under axial compression, the following are the conclusions were made.

1) The increase in length of steel sections will results in decrease of allowable buckling load on the columns.
2) The sections provided with stiffener are failed due to distortional buckling whereas the plane anglesections are failed due to local buckling.
3) For the given height, occurrence of global buckling is not possible.
4) When the slenderness ratio increases, the load carrying capacity of the compression members weredecreased.

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