Behaviour of Cold-Formed Steel Built-up I Section Under Bending

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***Abstract* - This paper presents an experimental and numerical investigation on the bending strength and behaviour of cold-formed (CF) steel built-up flexural members. Eight specimens in two groups, first group of four specimens with equal flanges and second group of four specimens with unequal flanges have been fabricated and experimented. The experimental results show the modes of buckling and their influence on the bending strength and behaviour of CF built-up I sections. The experimental results are also verified by simulating finite element models and analysed using FEM software ANSYS. The results obtained are in good agreement with the experimental results.**

Keywords - Cold-formed Built-up section, Thin walled beam, Local buckling, Flexural behaviour, Distortional buckling, Crippling

I. INTRODUCTION

Cold formed steel products are extremely used in building industry and range from purlins to roof sheeting and floor decking. These sections are extensively used in various engineering applications because of their high strength to weight ratio. Thin walled I sections are preferred mostly due to their light weight and full strength capacity.

Vlassov[1] presented the classical theory in this field excellently . Reviewing the present state of art one should mention the books presented by Bleich[2],Timoshenko[3], and Murray[4], in addition to a large number of papers based on linear analysis. Chajes[5], Yu[6], Bulson[7], Hancock[9], Pekoz [10,11] presented many papers on the stability of open cross-sections with solid webs. Based on the equilibrium at the deflected position of a structure Ganapathy Chettiar and Varghese[8] have developed a method to find the lateral buckling load of open-web frames and girders of any geometry and any support conditions. An experimental study on I-beams with slender sections has been carried out by Kubo and Fukumoto[12] and presented the interaction behaviour of local and lateral-torsional buckling in the inelastic buckling range. They also found that the beams with intermediate length undergo combined local and lateral-torsional buckling whereas the long beam fails only by lateral-torsional buckling.

Put, Pi and Trahair[13] conducted lateral buckling strength tests on C and Z beams and summarized that local or distortional buckling occurs in the compression element. Yu and Schafer[15] presented the testing plan to restrict distortional buckling and a simple method to generate local buckling in C and Z beams. Yu and Schafer[18] presented another method to generate distortional buckling in cold formed steel C and Z beam and they also found that large strength reduction occurs as the distortional buckling is initiated instead of local buckling. In case of latticed steel members, as a bar with circular cross-section is very stiff torsionally and flexurally and as the full section of the bar is continuous at the joints where they are also welded, the system of web bars provides a high torsional and flexural stiffness at joints. In this, the load transmission relies solely on the weld connection, resulting in a discontinuous load-transfer mechanism. Heavy stress concentration is induced at the welded locations.

Though many studies have been performed on buckling of cold formed steel profiles, few studies have been made on latticed / built-up cold-formed beams and remain largely unexplored.

The literature on built – up CF Sections are very minimum and are available for compression members[14,16,17]. Whereas the study on cold formed steel built up sections under bending is very rare. Hence in this investigation an attempt has been made to study the behaviour of CFS built – up flexural members and to find the feasibility of such sections in the field of construction. Thin walled built-up I section beams are economical with same flexural rigidity when compared to thin walled solid web sections. Though this type of sections results in economic type of construction, presently the usage of such members are limited because of the complexity involved in the analysis and prediction of their behaviour become tedious. As this type of sections with open cross section are susceptible to multiple modes of buckling when they are connected together by a system of continuous bent bar with circular cross sections, the behaviour of the beam elements may be governed by local buckling in the component plates, distortional buckling of the outstanding legs and the local buckling of web element.

The Research activities presented in this current paper are focused on flexural behaviour of cold-formed steel latticed members under static lateral loading is a part of the ongoing research activity. This paper provides the outcome of the first experimental phase of the research devoted to study the behaviour of the latticed cold-formed steel beams.

II. TEST SPECIMEN

Two types of built up CFS I Sections with equal and unequal flanges have been fabricated and experimented. The specimens are fabricated from 1.2mm thick steel sheets whose yield stress is 230 N/mm^2 and young's modulus of 1.98 x 10^5 N/mm^2. The specimens are built-up using four numbers of Cold Formed angles with lip in the flange alone. Out of four angles two are provided in the top flange and two at the bottom flange which act as the main chords of the member. These main chords are held in position by using a secondary member of 6mm diameter bar bent in the form of sinusoidal wave form of angle 45°. The main chords and the secondary members are welded at the point of contact by spot welding. The cross section of the specimen is shown in the Fig.1 and the geometric properties of the section are tabulated in the Table 1. The series are designated as UEI – top width – thickness for unequal I sections and EI – top width – thickness for equal I sections. All the eight specimens were tested under two point loading. UEI – 40 – 1.2 is an unequal I section fabricated without stiffener under loading points so as to study the behaviour in bearing. EI – Equal flange I sections were fabricated with 2mm thick stiffener plates to investigate the influence of bearing stiffeners. Other specimens were fabricated to study the local and distortional buckling of beams by strengthening the stiffeners against bearing failure. Two stiffener plates one on either side is provided at the points of loading and at the points of support.

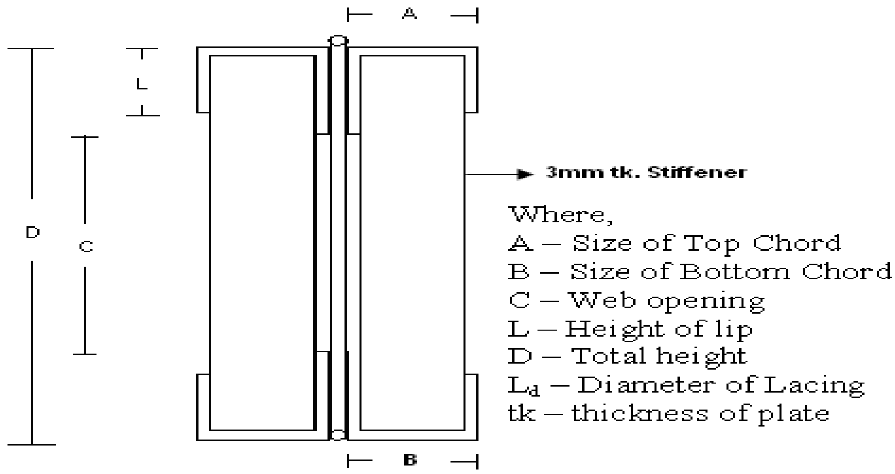

Fig. 1 Cross Section of the Specimen

Table I Geometric Properties of the Sections

S. No	Description of Specimen	(A) mm	(B) mm	(C) mm	(L) mm	(D) mm	(ld) mm	(tk) mm
1.	UEI – 40 – 1. 2	40	20	30	15	90	6	1.2
2.	UEI – 40 – 2. 0	40	20	30	15	90	6	2.0
3.	UEI – 35 – 1. 2	35	20	35	15	90	6	1.2
4.	UEI – 30 – 2. 0	30	20	40	15	90	6	2.0
5.	EI – 40 – 1. 2	40	40	10	15	90	6	1.2
6.	EI – 40 – 2. 0	40	40	10	15	90	6	2.0
7.	EI – 30 – 1. 2	30	30	30	15	90	6	1.2
8.	EI – 30 – 2. 0	30	30	30	15	90	6	2.0

III. EXPERIMENTAL INVESTIGATION

The experimental work was conducted on a 50kN capacity self-straining frame. The Specimens were experimented under simply supported end condition with two points loading. Deflectometers were placed at three positions namely 1/3rd distance, mid span and at support. Strain readings were taken in the top compression flange and in the lip of the compression flange. A Proving ring of 50kN capacity was used to measure the loading. The load and corresponding deflection was observed. Loading was gradually applied till the specimen become unstable. The procedure was repeated for all the eight specimens. The experimental set up is shown in Fig. 2. The modes of failure are noted for every specimen. Some of the tested specimens are shown from Fig. 3 to Fig. 6. The total critical moments, total ultimate moments and bending stress are presented in Table II.

Fig. 2 Test Setup

Fig. 3 Distortional Buckling

Fig. 4 Wobbling of Bottom Web

Fig. 5 Local Buckling

Fig. 6 Flexural Buckling

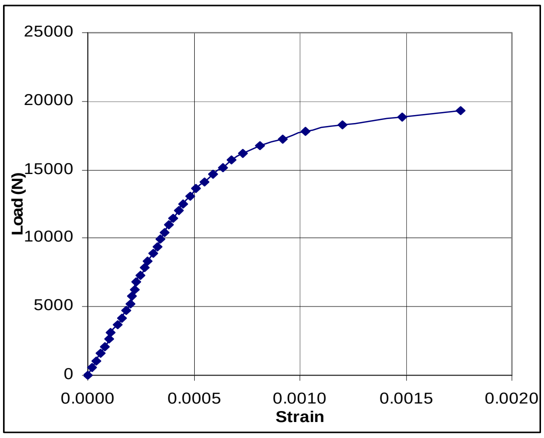

Fig. 7(a) UEI – 40 – 2.0

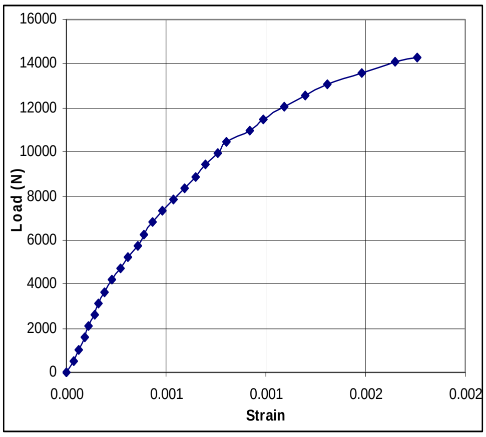

Fig. 7(b) UEI – 35 – 1.2

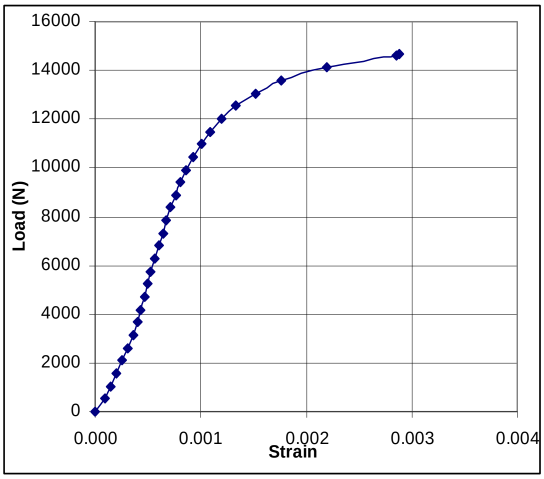

Fig. 7(c) UEI – 30 – 2.0

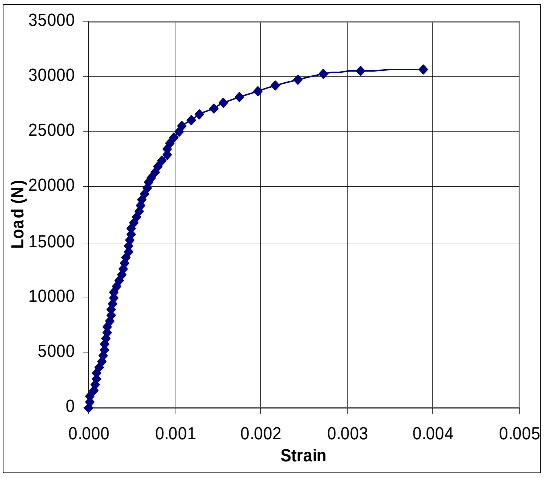

Fig. 7(d) EI – 40 – 2.0

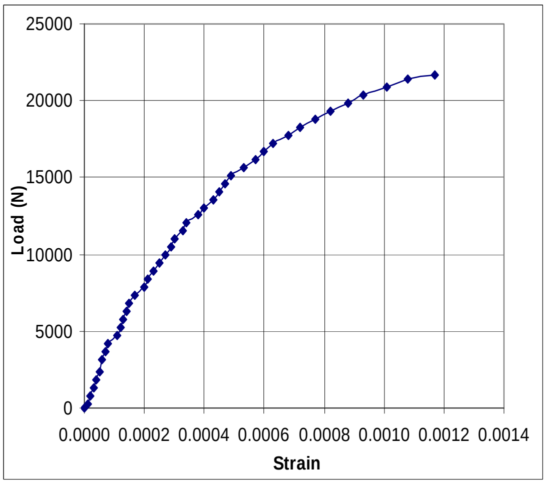

Fig. 7(e) EI – 30 – 2.0

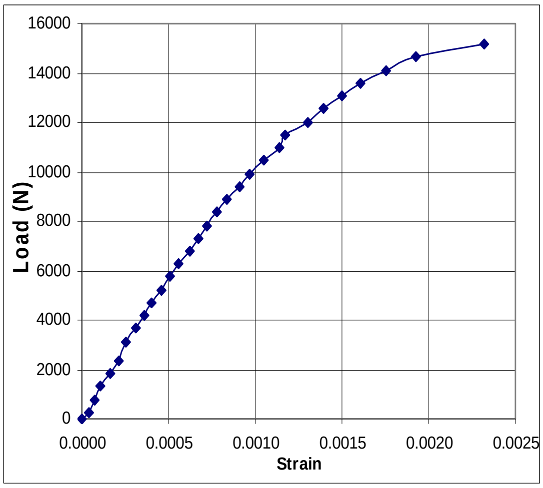

Fig. 7(f) EI – 30 – 1.2

Fig. 7 Load Vs. Strain Curves

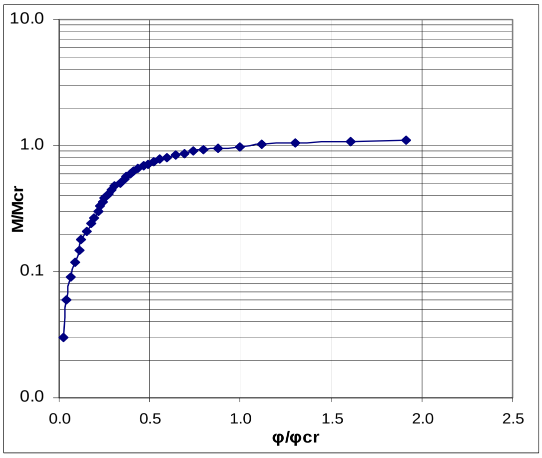

Fig. 8(a) UEI– 40 - 2.0

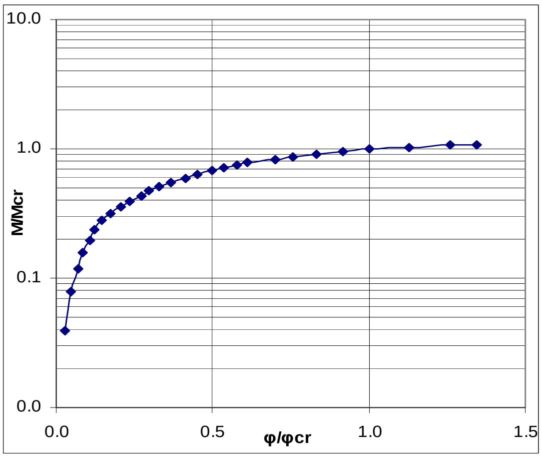

Fig. 8(b) UEI – 35 – 1.2

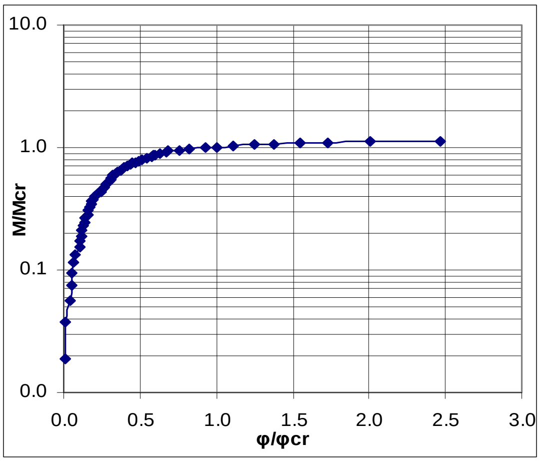

Fig. 8(c) EI – 40 – 2.0

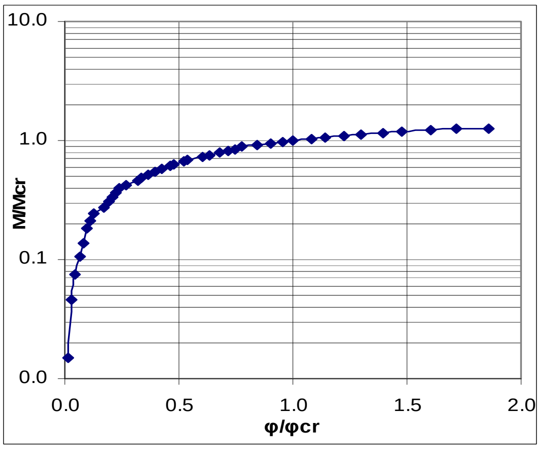

Fig. 8(d) EI – 30 – 2.0

Fig. 8 Moment Vs. Curvature Curves

Table II Experimental Results

S. No.	Description of Specimen	Critical Moment kNm	Section Modulus mm^3	Critical Stress σ_{cr} N/mm^2	Ultimate Moment kNm	Ultimate Stress σ_{ult} N/mm^2	$\frac{\sigma_{cr}}{\sigma_{ult}}$	Mode of Failure
1.	UEI – 40 – 1. 2	2.76	9394	146.91	3.75	199.60	1.36	B
2.	UEI – 40 – 2. 0	5.25	15372	170.76	5.70	185.40	1.09	D
3.	UEI – 35 – 1. 2	3.78	9312	202.96	4.20	225.50	1.11	F
4.	UEI – 30 – 2. 0	3.96	15291	129.50	4.50	147.10	1.14	D
5.	EI – 40 – 1. 2	3.75	13927	134.63	6.60	236.95	1.76	LS
6.	EI – 40 – 2. 0	8.25	22962	179.64	9.30	202.51	1.13	F
7.	EI – 30 – 1. 2	3.00	11708	128.10	3.90	166.60	1.30	L
8.	EI – 30 – 2. 0	5.17	19327	133.90	6.60	170.75	1.28	L

B – Bearing Failure, D- Distortion, LS – Local Buckling of Stiffener, F- Flexural Buckling, L – Local Buckling

IV. FEM ANALYSIS USING ANSYS

An approximate idealised mid-line model with four noded shell – 143 element, for plate and 3D element for lacing are chosen. A non – linear analysis with large deflections on has been carried out. The model is force- loaded and the line search option is chosen for the analysis. All the sections fabricated for the experimental work (Table 1) were modelled and analysed. Failure mode shapes for some specimens are shown from Fig. 9 to Fig. 12 and the Critical moments and Ultimate moments are presented in Table 3.

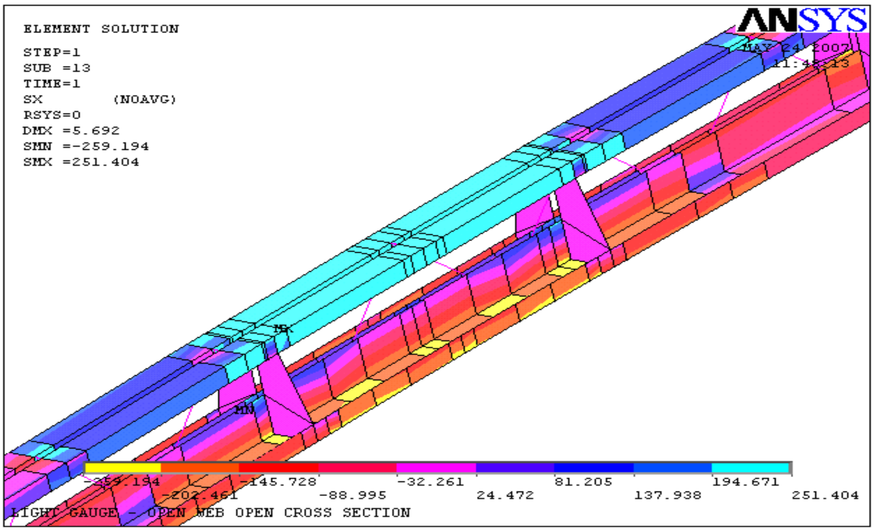

Fig. 9 Wobbling in Web

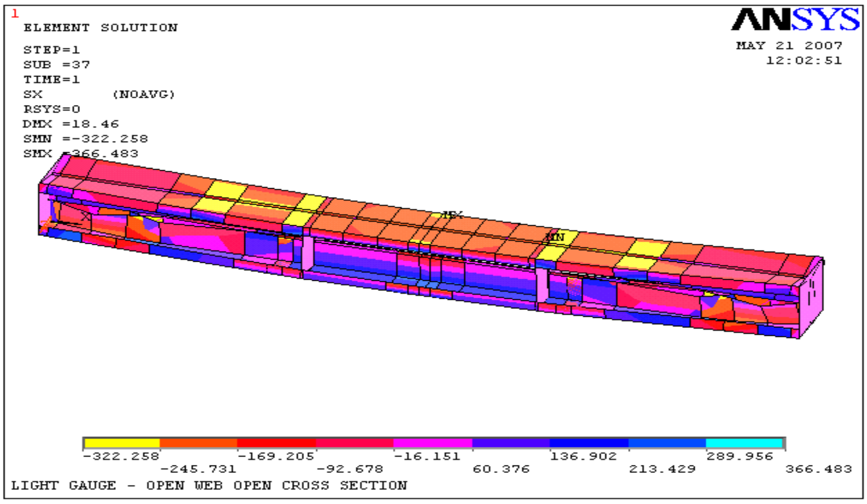

Fig. 10 Local Buckling

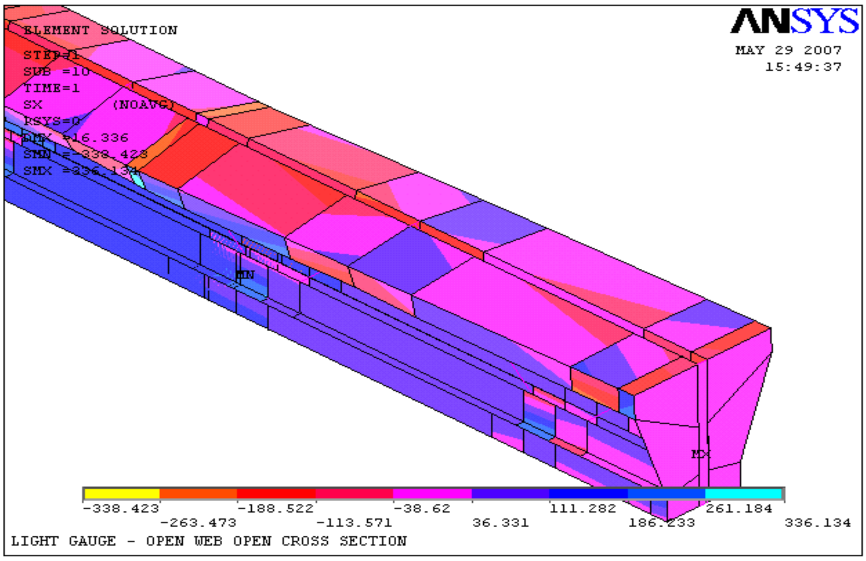

Fig. 11 Distortional Buckling

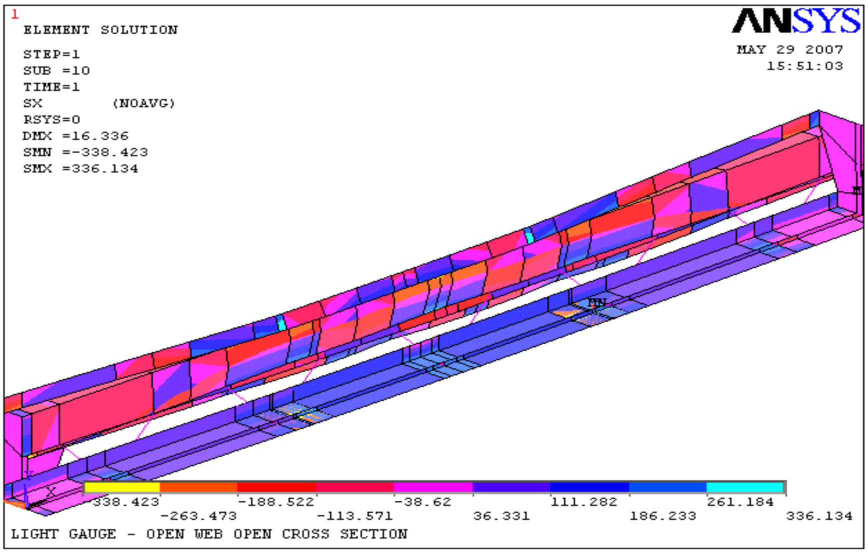

Fig. 12 Distortional Buckling

Table III Comparison of Experimental and ANSYS results

S. No.	Description of Specimen	Total Critical Moment (kNm)		Exp / ANSYS	Total Ultimate Moment (kNm)		Exp / ANSYS
		Exp.	**ANSYS**		**Exp.**	**ANSYS**	
1.	UEI – 40 – 1. 2	2.76	2.82	0.98	3.75	3.56	1.05
2.	UEI – 40 – 2. 0	5.25	5.40	0.97	5.70	6.60	0.86
3.	UEI – 35 – 1. 2	3.78	3.60	1.05	4.20	4.50	0.93
4.	UEI – 30 – 2. 0	3.96	3.75	1.06	4.50	4.80	0.94
5.	EI – 40 – 1. 2	3.75	3.90	0.96	6.60	6.75	0.98
6.	EI – 40 – 2. 0	8.25	8.10	1.02	9.30	9.00	1.03
7.	EI – 30 – 1. 2	3.00	3.23	0.93	3.90	4.20	0.93
8.	EI – 30 – 2. 0	5.17	5.4	0.96	6.60	6.75	0.98

V. RESULTS AND DISCUSSION

The failure models are mixed bearing, local, distortional and flexural. The experimental failure modes are in good agreement with ANSYS results. Load Vs. Deflection curve shows that both exp and ansys are in good agreement. Bearing failure occurred in UEI – 40 -1.2 under the loading points followed by local buckling of the elements of top chord members. EI series failed by buckling of stiffeners followed by mixed local and distortional bucking of compression chords. Wobbling effect as a sine wave is found in all the unstiffened elements (Lip and web elements). In the compression zone, the individual top chord members were distorted. UEI series failed by mixed local and distortional buckling of the individual chords. Equal Flange I-section carried approximately 40% more than that of unequal flange I – section. The comparison shows that member strength is closely related to the failure pattern. For example the members (EI– 40– 2.0, UEI–35–1.2) failed by flexural buckling exhibited higher bending stress closer to the yield stress and ultimate stress is reached as soon as the critical stress is reached, whereas those (EI – 30–1.2, EI–30– 2.0) failed by local buckling in the lip and flange were unable to reach their yield stress. The members(UEI-40-2.0, UEI-30-2.0) failed by distortional buckling exhibited very low post buckling reserve strength, whereas those failed by local buckling (EI-40-1.2) showed high post buckling reserve strength.

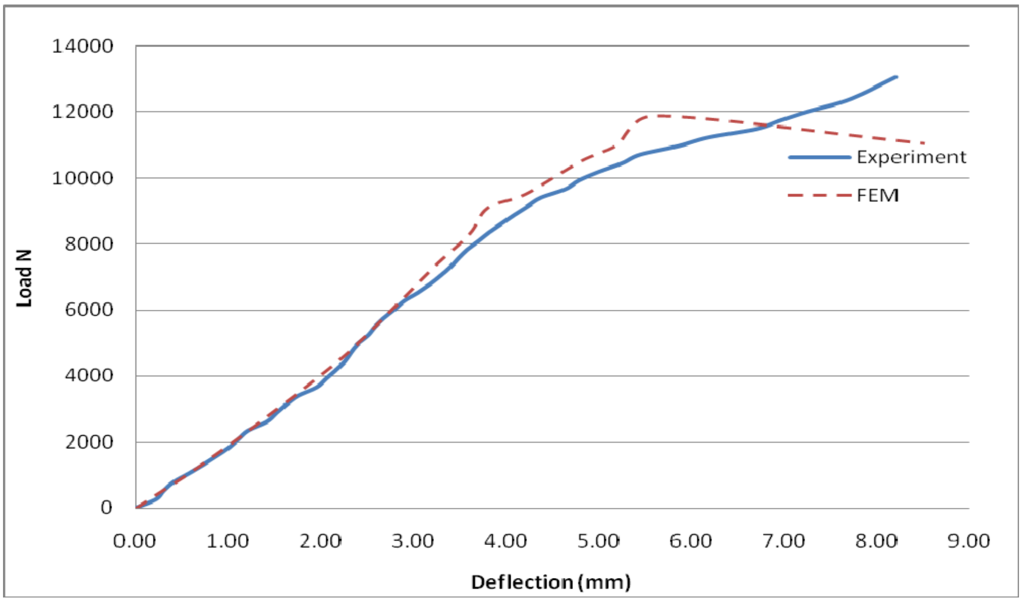

Fig. 13 Load Vs Deflection Curve (UEI 40 - 1.2)

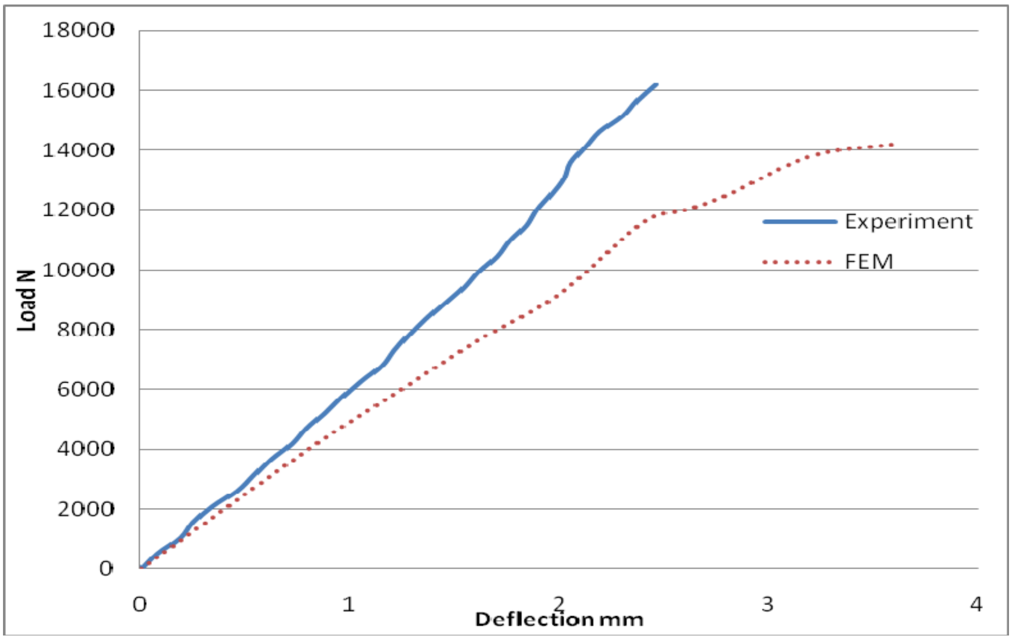

Fig. 14 Load Vs Deflection Curve (UEI 35 - 1.2)

VI. CONCLUSIONS

The experimental and numerical analyses on the bending strength and behaviour of cold-formed steel built-up I section flexural members have been investigated. The conclusions from the investigations are as follows

- It is observed that each chord member bend individually. This shows that this type of members does not bend in a single wave as a whole as in the case of solid cross- section, but the constituent members bend individually.
- The web element also contributing appreciably to the strength.
- Specimen failed under distortional buckling have little post buckling reserve strength whereas specimen failed under local or bearing have more post buckling reserve strength.
- To avoid bearing failure vertical stiffeners are required at support and at the loading points.
- By stiffening the web element of each chord member the capacity of the beam is further improved.
- Local buckling, distortional buckling and interaction between local and distortional buckling were observed. The FEA predictions are generally in good agreement with the experimental buckling modes. The results show that the buckling mode has greater influence on the strength of the specimens.

- The comparison between the experimental results and the FEA predictions prove that the finite element analysis is a reliable tool to get quite accurate results in a reasonable amount of time. So the parametric study can be performed by the FEA to investigate the behaviour of Cold Formed Steel built up sections.

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