Pure Bending study on Cold Formed Steel- Latticed Built-up I beam

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***Abstract* - Steel is a worldwide construction material, which is a combination of Iron and Carbon. There are two types of steel sections; one of the sections is Cold Formed Steel (CFS) section, which are used mainly in lightweight building applications. Features of CFS are lightness, high strength and stiffness, mass production and easy erection, economy in transportation and handling, non-combustibility, recyclable material. The advantages of CFS are that the yield strength of the steel is increased in cold forming due to the consequence of cold work into strain - hardening range. The predominant zones were increased where the metal is bent by folding. The effect of cold working is thus to enhance the mean yield stress by 15% - 30%. The yield stress may be enhanced by a minimum of 15%. In this project a Built-up I section has been created by using 4 angles connected back-to-back. The stiffeners were provided at the ends and 1/3rd of the span (loading point). Lacings were used to connect the angles and to improve the moment carrying capacity of the section. The theoretical calculation as per North American Specification of Cold Formed Steel (AISI S-100:2007) has been investigated and modes of failure has been observed. Additionally experimental study has been performed for specimens fabricated by varying different sectional properties and compared with the theoretical results.**

***Index Terms* - Steel, Cold Formed Steel, Lacing, Direct Strength Method, Experimental Investigation.**

I. INTRODUCTION

Steel is a worldwide construction material which is a combination of Iron and Carbon stated by Paul W Griffin & Geoffrey P Hammond (2019). There are two types of steel sections used in the construction industry, they are Hot rolled sections and Cold formed sections as stated by Leroy Gardner et al. (2019). Hot rolled sections are mainly used as structural members where heavy loads are to be carried whereas Cold formed steel sections are used mainly in light weight building applications.

Cold-formed steel (CFS) section is a product which are made by rolling or pressing thin gauges of steel sheets into desired sections in order to meet its purpose in construction. CFS products are created by the working of thin steel sheets using stamping, rolling or presses to deform the steel sheets into a proper product which can be used as a structural components like columns, studs, beams, floor decking, built up sections and other any parts of the structure. The manufacturing process of CFS section is differ from Hot rolled steel which showing alternate in properties in comparison of both the sections.

1.1 FEATURES OF CFS

The cold formed steel sections are widely used in the market for construction and other aspects. They are mainly popular due to the following reasons are:

- Lightness
- High strength and stiffness
- Mass production and easy erection
- Non-shrinking and non-creeping at ambient temperatures
- Formwork not needed
- Uniform quality
- Economy in transportation and handling
- Non-combustibility
- Recyclable material

1.2 ADVANTAGES OF CFS

The yield strength of the steel is increased in cold forming due to the consequence of cold work into strain- hardening range. The predominant zones were increased where the metal is bent by folding. The effect of cold working is thus to enhance the mean yield stress by 15% - 30%. The yield stress may be enhanced by a minimum of 15%. The advantages of cold rolled sections, as compared with their hot rolled counterparts as stated few by Benjamin W Schafer (2019) and Shahriar Afkhami et al. (2019) are as follows:

1. The Cross-sectional shapes of CFS are formed to close tolerances and they can be frequently repeated for as extensive as necessary.
2. For desired shape and length, the cold rolling is engaged.
3. Pre-galvanised or pre-coated metals can be formed, so that high resistance to corrosion, besides an attractive surface finish, can be achieved.
4. All conventional jointing methods, (i.e. riveting, bolting, welding and adhesives can be employed.
5. Cold rolled gives the high strength to weight ratio.
6. For transport and erection is easy due to light in weight.

1.3 APPLICATIONS OF CFS

CFS can be grouped into the major areas of use such as:

1. Compression members.
2. Corrugated and curved panels.
3. Flexural members and purling.
4. Composite and plasterboard construction.
5. Storage racks.

1.4 LATTICED BUILT-UP SECTION

Latticed built-up sections are made using combination of structural sections which are connected with diagonal lacing bars. Latticed built-up sections are used whenever a single section is not sufficient to carry the load.

Generally, latticed built-up sections are used

1. To resist axial compression - by means of struts like stiffeners.
2. To resist axial tension - by means of ties like Lattice and
3. To resist bending - by means of beams profile.

The built-up members are formed by connecting two or more cold formed steel members together, such as I section Member built-up by connecting two channel sections back to-back or four angle section back-to-back with intermediate and end stiffeners as shown in Fig.1.1.

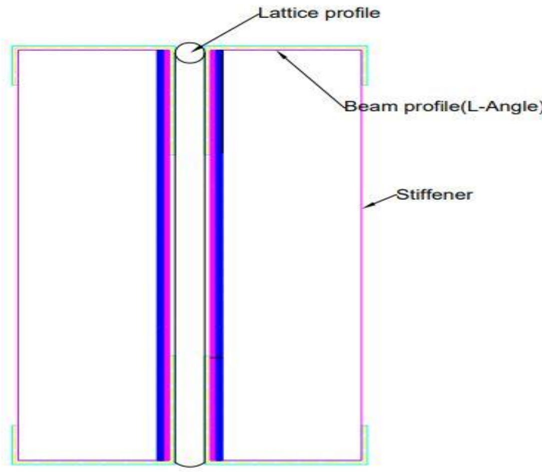

Figure.1.1 Latticed Built-up cross section

II. LITERATURE REVIEW

Zavelani & Faggiano (1985) started using Cold-Formed Steel sections for latticed transmission towers. They designed the transmission based on the AISI "Specifications for the Design of Cold-Formed Steel Structural members". They used all shapes of cold - formed sections except plain angles and presented experimental results reflecting the correlation between the results and design recommendations.

Davies (2000) has analysed the inelastic load carrying capacity of CFS beams with stiffened compression flanges has been investigated. A failure criterion based on the ultimate compressive strain is developed experimentally for beams under uniform moment and moment gradient. In any limit analysis, a trial-and-error procedure is generally this trial-and-error step. During the course of this investigation, the webs of test specimens were purposely kept stocky to avoid web failure. In the web slenderness ratio of flexural members is large, buckling may imitate the web into a number of modes, causing permanent failure.

Trahair (2000) makes available a review of condition as the knowledge on top of the lateral type buckling which strengthens the unsheathed CFS beams. On the other hand, there are many studies of inelastic buckling of thick-walled cold formed beams either theoretical or experimental and there is a need to conduct experiments so that data can be obtained for assessing design methods. Many thin-walled beams fail by local buckling of compression flange after excessive lateral deflection but design methods do not reflect this and there is a scarcity of experimental data.

B. W. Schafer, A. Sarawit and T. Pekoz (2006) has investigated the behaviour and provided recommendations on the design of open cross-section thin-walled cold-formed steel members that employ complex stiffeners. Complex stiffeners 13 were formed when the free edge of an open section was folded multiple times, as opposed to simple stiffeners which employ a single fold. Simple stiffeners become ineffective when long lips are required to stabilize the flange, as the lip itself initiates the instability. In the elastic buckling regime, complex stiffeners were showed to hold a distinct advantage in local buckling over simple stiffeners. For complex, or sometimes referred to as compound stiffeners, most of the existing work has focused on cold-formed steel storage rack posts, which use variety of different complex stiffeners. The elastic buckling behaviour of members with complex stiffeners was accomplished by numerical methods. General purpose finite element method solutions, or more specialized methods such as the finite strip method or generalized beam theory. Lau and Hancock 1987 extended the work with experiments and a closed-form solution for estimating the elastic critical distortional stress. More general strength expressions for distortional buckling Hancock et al. 1994were adopted by the Australian/New Zealand standard AS/NZS 4600 AS/NZS 1996 and still remain today as the codified design method for distortional buckling.

Long-Yuan Li (2009) presented a study on the calculation of the critical stress of distortional buckling of cold- formed sigma purlins using EN1993-1-3. The discussion is focussed on the determination of the spring stiffness of the stiffened element, a problem which has not yet been addressed in most design codes. Different support conditions at both the tension and compression ends of the web are employed and their influences on the critical stress of distortional buckling of sigma purlins are investigated. Comparison with finite strip analysis indicates that the model having a fixed support for the tension end and a roller support for the compression end of the web provides the best fit to the finite strip analysis.

Lei Xu, (2009) had done the study involving finite element analysis to investigate the flexural strength of built-up box sections which have been extensively used in residential and commercial construction in North America. Both the Lightweight Steel Framing Design Manual (CSA, Supplement 2004 to the North American specification for the design of cold-formed steel structural members, S136, Canadian Standard Association, 2004) and cold-formed Steel Framing Design Guide (AISI, Cold-Formed steel framing design guide, American Iron and Steel Institute, 1st ed., 2002, 2nd ed., 2007) have recommended that the flexural strength and moment of inertia of the built-up sections to be taken as the sum of the two components based on deflection compatibility of the components. However, this design approximation has yet to be justified by experimental or numerical study especially for the cases of eccentric and concentric loading.

Haiming Wang, Yaochun Zhang (2009) stated that in practical situations, flexural members are subjected to a variety of moment gradients. In design, moment gradient is considered by an equivalent moment factor in the calculation of lateral–torsional buckling, but there is not a similar factor in calculation of local buckling or distortional buckling. In these tests, none of the specimens was failed in lateral–torsional buckling, so the equivalent moment factor is not usable. The test bending strengths under the two bending states are compared. All of the bending strengths under non-pure bending are higher than which under pure bending. For the short-inclined edge stiffener specimen, the bending strength increased more than 17% under moment gradient compared with under pure bending. Whereas for the long upright, long inclined and the two complex edge stiffener specimens, the increased magnitude is so small that no one exceeds 5%. The short upright edge stiffener specimen seems special that the increased magnitude is 6.09% which is greater than the two long and two complex edge stiffener specimens and also far smaller than the short-inclined edge stiffener specimen. It is well known that, the half-wavelength of local buckling is shorter than that of distortional buckling. For one gradient moment, the moment varies little in a range of one-half wavelength of local buckling but varies considerable in a range of one half-wavelength of distortional buckling. So, the moment gradient has only a minor influence on local buckling but has great influence on distortional buckling. For the short-inclined edge stiffener specimen, only distortional buckling was observed in both the pure bending and non-pure bending test. Moment gradient has great influence on distortional buckling and induces the bending strength of non-pure bending test increased more than 17% compared with in pure bending test. For the two long and two complex edge stiffener specimens, local buckling was observed in both the two bending tests, distortional buckling was also observed but not obvious, local buckling was the principal failure mode. Moment gradient has only a minor influence on local buckling, so the bending strengths under the two bending states vary little, less than 5%. For the short upright edge stiffener specimen, only distortional buckling was observed in pure bending test, but interaction between local and distortional buckling was observed in non-pure bending test. Because of the detrimental effect of interaction between local and distortional buckling, the bending strength of non-pure bending test merely increased 6.09% compared with in pure bending test. The test results also provide strong evidence that moment gradient has only a minor influence on local buckling, but has great influence on distortional buckling and cannot be neglected.

Cheng Yu & Weimimg Yan (2011) Proposed a design method, based on effective width method, for determining the nominal distortional buckling strength of typical cold formed steel C and Z sections subjected to bending. The method can be integrated into the classic effective width design provisions specified in AISI S100, and it allows the convention design approach to cover more comprehensive limit states.

Jung Kwan Seo, Mahen Mahendran, (2011) stated that for,

Section moment capacity of flexural members without holes:

The prediction and calculation of the section moment capacity (M_s) of flexural members rely on American Iron and Steel Institute (AISI) design rules based on the effective widths of stiffened elements. Flexural members without holes are not subject to lateral torsional buckling, and thus their section moment capacities can be computed using these rules. The effective yield moment based on section strength, M_s, is determined as follows. $M_s = S_eF_y$, where F_y is the design yield point determined in the AISI and Se the elastic section modulus of an effective section calculated relative to extreme compression or tension fibre at F_y ($f=F_y$).

Section moment capacity of flexural members with holes:

Flexural members with holes are also not subject to lateral torsional buckling, and hence their section moment capacity can also be computed using the AISI design rules with effective section modulus, S_e, at $f=F_y$. The section moment capacities (M_s) of 12 LSB sections were calculated using the AISI design rules. The rules were also employed to determine these capacities without consideration of the aforementioned local buckling effect and using the full width of the web component [$=(h-d_o)/2$]. The corners of the LSBs were also included, but their effect was minimal.

Cao Hung Pham a, Gregory J. Hancock (2012) Provided the solutions to the elastic buckling of whole channel sections including flanges and lips where the sections are loaded in pure shear parallel with the web. In this investigation, The channel consists of a web depth of 200 mm, a flange width of 0.01 mm to 160 mm, a lip size of 0.01 mm to 20 mm, all with thickness of 2 mm was used. The very small value of 0.01 mm in both flange width and lip size, although impractical, has been used to allow the limiting condition of zero flange width and unclipped channel to be approached. The member is subdivided into 40 longitudinal strips which include 16 strips in the web, 10 strips in each flange and 2 strips in each lip. The length of the member studied varies from 200 mm to 2000 mm. The elastic buckling analyses of thin-walled channel sections in pure shear have been implemented by means of the Spline Finite Strip Method (SFSM) to determine the elastic buckling loads (Vicar) of the sections. The shear buckling coefficients (kiva) of the web for design of a section was back-calculated from the shear buckling loads. The main variables were the flange widths, member lengths and lip sizes. The boundary conditions at two end sections were simply supported. The results of the analyses plotted in the format of interaction charts was given as design guidelines for designers to predict the elastic buckling shear coefficient (kiva) without using the Spline Finite Strip Method (SFSM) software. The Spline Finite Strip Method (SFSM) was a development of the semi analytical finite strip method originally derived by Cheung. It uses spline functions in the longitudinal direction in place of the single half sine wave over the length of the section and has been proven to be an efficient tool for analysing structures with constant geometric properties in a particular direction, generally the longitudinal one. The advantage of the Spline Finite Strip Method was that it allowed more complex types of loading and boundary conditions other than simple supports to be easily investigated and buckling in shear was also easily accounted for. Lipped channel sections with variable member lengths, flange widths and lip sizes were analysed by the Isoperimetric Spline Finite Strip Method program. The shear flow distribution resulting from a shear force parallel with the web is used for analyses. By varying the member lengths, flange widths and lip sizes, the analysis results showed that the flanges with lips can have a significant influence on improving the shear buckling stress of thin-walled channel sections. When the channel member was long, the channel member with a very narrow flange may buckle in a twisting mode. As the flange width increases, the buckling mode will change to the distortional buckling mode at a certain flange width. When the flange was wide enough to provide elastic torsional restraint to the web, the channel member will buckle mainly in a local buckling mode.

Vijayasimhan M, Marimuthu V, Palani G.S. and Rama Mohan Rao P (2013) carried out the Comparative Study on Distortional Buckling Strength of Cold Formed Steel Lipped Channel. The Indian code for cold-formed steel design, IS 801 was revised during 1975, which is in line with 1968 edition of AISI standard. Bureau of Indian standards is in the process of revision of IS 801 to catch up with the latest developments and design methods with the other codes of practices in the world. As a background for the development of codal provisions, the design provisions developed in the various codes of practices have been reviewed and a comparative study has been carried out in this investigation. In this investigation following codes were studied:

1. AISI Standard – North American specification for the design of cold formed steel structural members.
2. BS 5950-5:19987.
3. IS 801 Code of practice for use of cold formed light gauge steel structural members in general building construction.
4. Eurocode 3
5. Australian/New Zealand standard.

Design strength of each cross section was calculated by using the provisions given in all codes. For easy comparison the values predicted by various codes were normalized with experimental. It was found that, IS 801 over predicts the strength by about 30% due to the neglection of distortional mode of buckling and DSM closely matches with the experimental results. The DSM variation was about 4%. British code also has good correlation with experimental values. With this it can be concluded that Direct Strength Method of Design may be considered for possible incorporation in the IS 801 revision. As the DSM, which uses the load factors corresponding to different buckling modes and their interactions, predicts the flexural strength of the lipped channel section closer to the experimental results, parametric study had been conducted on distortional buckling strength of the cross-section by varying the lip depth for selected specimens.

Sudha.K and Sukumar.S (2014) have investigated the Behaviour of Cold Formed Steel Built-up I Section under Bending. An experimental and numerical investigation on the bending strength and behaviour of cold-formed (CF) steel built-up flexural members have been studied. 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 section. 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.

P. Manikandan and S. Sukumar (2014) have investigated the behaviour of stiffened cold formed steel built-up sections with complex edge stiffeners under bending. This paper is concerned with the effect of stiffened element at the flange/web junction and complex edge stiffener in behaviour and strength of built-up cold formed steel flexural members. An extensive experimental investigation and a finite element analysis of stiffened built-up cold formed steel beam section with complex stiffeners under two-point loading is presented. A nonlinear finite element model is developed and verified against test results. All the results are compared with the design strength calculated using the north American iron and steel institute specification for cold formed steel structures (AISI: S100, 2700) following the validation, an extensive finite element parametric study is conducted to study the influence of a range of parameters, and the results are compared with the nominal design strength by AISI: S100, (2700) and suitable recommendation are made.

Srinath.T and Shanmugarajan.M (2016) investigated on effect of web opening on the bending behaviour of cold formed steel built-up 'I' section. Thin walled cold-formed steel members have wide applications in building structures. In cases where beams carry less moment it is uneconomical to use traditional hot rolled steel. Cold formed steel is an apt solution for this case. For a latticed cold formed steel flexural member, the moment carrying capacity may be affected mainly by local, distortional or lateral torsional buckling. In this paper, the impact of web opening on the flexural behaviour of Cold formed built-up I section under two-point loading is investigated for the simply supported end conditions. Experimental investigation has been carried out on 8 specimens by varying the thickness and depth of the built-up beam. Numerical investigations have also been carried out using finite element analysis software ANSYS13.0.it is observed that the member 50-50-250-1.2-2400 carries maximum moment while comparing 1.2mm thick specimens. This is because stiffener takes up load in this specimen. But while considering 2mm thick specimens, 50-50-200-2.0-2400 carries maximum moment. This shows that web opening to depth ratio of 0.5 would carry maximum moment. Also lacing failure is common in 2mm specimens and seen in 1.2mm specimen with maximum web opening. Even though a lateral displacement is arrested at support, lateral torsion is unavoidable for 2mm thick specimens of web opening 150mm and 200mm.

Martins et al. (2017) mooted numerical analysis on CFS segments with three dissimilar cross-sectional elements, with diverse conditions in addition to the theme of uniform bending. All the beams are displayed on the flange generated local buckling in addition to overcoming f_{crd} / f_{crl} relation ranges from between 0.5 and 2.00. A broad range of relations which yield stress in the direction of the non-significant buckling stress also covered in the experimentations. Numerical results will be used to evaluate the correctness of DSM (Direct Strength Method) while applied towards the beam's weakening due to limited distortional interactive method. The researchers accomplished that the existing DSM strengthened the curve intended distortional buckling be able to provide correct estimates while the local buckling pressure is perceptibly better than distortional buckling pressure in which case local distortional buckling interface be able to ignored. In both the stresses may be comparable in magnitude otherwise distortional buckling pressure go beyond the local buckling pressure, the authors optionally modify the DSM procedure on behalf of beams by means of the same taken on for columns support on the NDL or NLD works. Both the approaches are found to give good forecasts.

III. THEORETICAL INVESTIGATION OF LATTICED BUILT-UP I SECTIONS

3.1 SELECTION OF SECTION

For the Theoretical investigation, the latticed built up I section has been chosen, which consists of

- Angle
- Cover Plate
- Stiffener
- Lacing

Built-up I section has been created by using 4 angles connected back-to-back. The stiffeners were provided at the ends and 1/3rd of the span (loading point). Lacings were to connect the angles and to improve the moment capacity of the section.

Specimen	Thickness (mm)	Span (mm)	Overall depth (mm)	Diameter of Lacing (mm)	Depth of Lip (mm)
50-50-1.5-15-150-2400	1.5	2400	150	10	15
50-50-2-15-150-2400	2	2400	150	12	15
50-50-1.5-15-300-2400	1.5	2400	300	10	15
50-50-2-15-300-2400	2	2400	300	12	15

Table.3.1 Specimen details

3.2 AS PER NORTH AMERICAN SPECIFICATION FOR COLD FORMED STEEL STRUCTURAL MEMBERS (AISI S-100:2007) - THEORETICAL CALCULATION RESULTS FOR SPECIMENS

M_y - Initial Yield Moment

M_{ne} - Lateral Torsional Buckling resistance

M_{nl} - Local Buckling resistance

M_{nd} - Distortional Buckling resistance

M_u - Moment carrying capacity

Specimen	Moment carrying capacity M_u (kNm)	Modes of Failure
50-50-1.5-15-150-2400	11.46	Lateral Torsional buckling
50-50-2-15-150-2400	12.55	Local buckling
50-50-1.5-15-300-2400	26.43	Local & Distortional buckling
50-50-2-15-300-2400	32.33	Local & Distortional buckling

Table.3.2 Moment carrying capacity as per AISI S-100:2007

3.3 MODES OF FAILURE

Specimen	M_y (kNm)	M_{ne} (kNm)	M_{nl} (kNm)	M_{nd} (kNm)	M_u (kNm)
50-50-1.5-15-150-2400	15.64	11.46	9.73	7.88	11.46
50-50-2-15-150-2400	15.49	14.82	12.55	11.59	12.55
50-50-1.5-15-300-2400	32.53	29.85	26.43	25.62	26.43
50-50-2-15-300-2400	37.06	35.56	32.33	30.89	32.33

Table.3.3 Mode of failure as per AISI S-100:2007

4. EXPERIMENTAL INVESTIGATION

4.1. FABRICATION DETAILS

For experimental investigation 10 specimens of varying angle size, span and thickness as shown in Fig.4.1 to 4.5 has been taken and the results are compared with theoretical results.

Specimen	Thickness of Plate (mm)	Thickness of Stiffener (mm)	Diameter of Lacing (mm)	Depth of Lip (mm)
50-50-1.5-15-150-2400(O3)	1.5	5	10	15
50-50-2-15-150-2400(O4)	2	5	12	15
50-50-1.5-15-300-2400(P1)	1.5	5	10	15
50-50-2-15-300-2400(P2)	2	5	12	15

Tab.4.1 Specimen details

Fig.4.1 Fabricated specimens - P series

Fig.4.2 Fabricated specimens - O series

4.2 TESTING OF SPECIMENS

After the completion of setup, the load is applied gradually at the loading points until the specimens fails. The deflections at every 50N (rate of loading = 0.05kN) load at various distances were noted. Mode of failure of the specimens were noted. The graph is plotted between Load vs Deflection. The moment carrying capacities of the tested specimens were calculated.

4.3 SPECIMEN - FAILURE MODES

O3 - 50-50-1.5-15-150-2400

Fig.4.3 Deformed Shape of O3 specimen

O4 - 50-50-2-15-150-2400

Fig.4.4 Deformed Shape of O4 specimen

P1 - 50-50-1.5-15-300-2400

Fig.4.8 Deformed Shape of P1 specimen

P2 - 50-50-2-15-300-2400

Fig.4.5 Deformed Shape of P2 specimen

4.4 EXPERIMENTAL RESULTS

Sl. No.	Specimen	Ultimate Load (kN)	Ultimate Moment (kNm)	Mode of Failure
1	50-50-1.5-15-150-2400	17.1	13.67	Lateral Torsional buckling
2	50-50-2-15-150-2400	22.2	17.75	Local buckling
3	50-50-1.5-15-300-2400	35.8	28.63	Local & Distortional buckling
4	50-50-2-15-300-2400	48	38.39	Local & Distortional buckling

Tab.4.6 Experimental results

4.8 LOAD VS DEFLECTION GRAPH

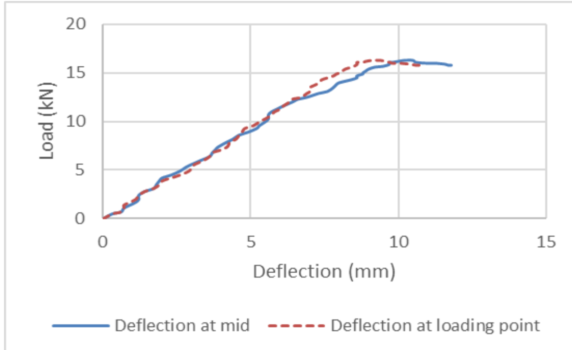

Fig.4.7 Load vs Deflection of O3

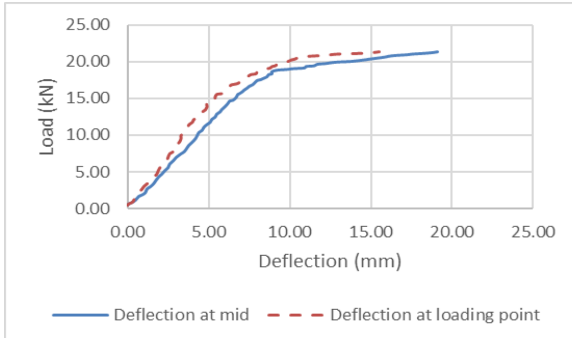

Fig.4.8 Load vs Deflection of O4

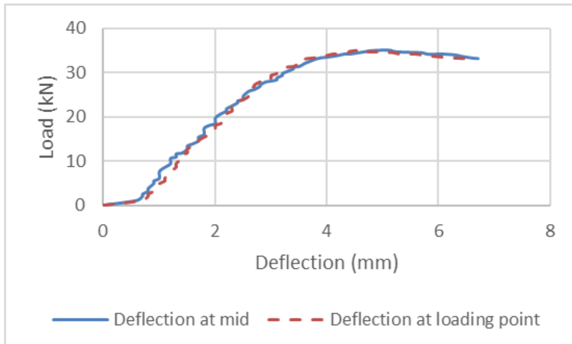

Fig.4.9 Load vs Deflection of P1

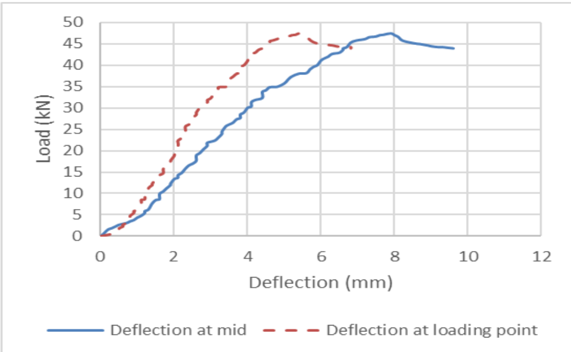

Fig.4.10 Load vs Deflection of P2

4.9 OBSERVATION & COMPARISON ON TESTING

As discussed above 4 specimens of different sectional properties has been tested and vividly taken into consideration for evaluation as these specimens resembles nearly same properties to one another but the response was different to each specimen under loading conditions. Therefore, with the reference of dimensional properties like span length, width of angle, overall depth of specimen & thickness of CFS sheet, the specimens were categorized and compared the results and presented.

Comparison based on Span length – 2.4m (O & P)

As discussed above, a total of 4 specimens under 2.4m span has been categorized as each specimens shows different responses like failure mode, peak deflection and peak load under loading condition using actuator. The below showing Fig. 4.26 depicts the representation Load vs Deflection graph of specimens (O3, O4, P1, P2).

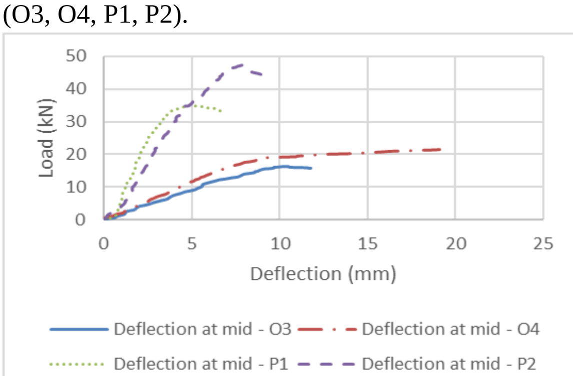

Fig.4.26 Load vs Deflection – Comparison of 2.4m specimen

As shown in graph the specimens of P series give higher load carrying capacity compared to O series this is due to increase in overall depth of P series specimens. Load carrying capacity is higher for P2 since thickness of CFS sheet is 2mm when compared to P1 of thickness 1.5mm. As in discussing about deflection, O4 shows larger deflection as overall depth (150mm) and thickness (2mm) is comparatively lesser to other specimens as it was been easier to fail due to thickness and depth of section. The failure mode (Fig.4.6 to 4.9) for P series shows combination of failure like starting with local bucking and continues to distortional buckling. O3 specimen shows lateral torsional failure at top flange portion near to one side of loading point but O4 specimen shows local buckling failure mode at mid of top flange portion. Even though the load carrying capacity of P series is good its post buckling resume strength is less compared to O series.

5. RESULTS AND DISCUSSION

5.1 COMPARISON OF RESULTS

Specimen details	Mu_{EXP}	Mu_{DSM}	Mu_{EXP}/ Mu_{DSM}
50-50-1.5-15-150-2400	13.67	11.46	1.19
50-50-2-15-150-2400	17.75	12.55	1.41
50-50-1.5-15-300-2400	28.63	26.43	1.08
50-50-2-15-300-2400	38.39	32.33	1.19

Tab.5.1 Comparison of Experimental and Theoretical results

5.2 COMPARISON OF CHARTS

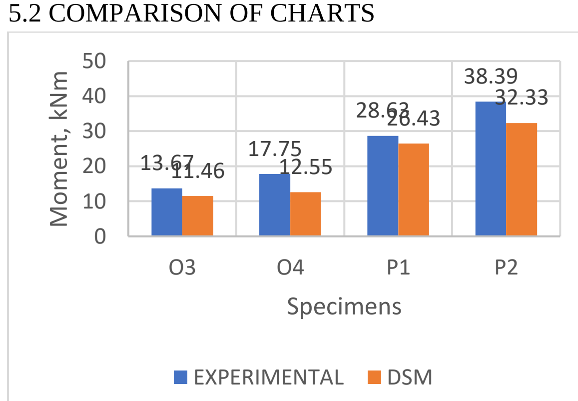

Fig.5.1 Comparison of Experimental and Theoretical results

5.3 DISCUSSION

1. Comparison of Theoretical investigation and Experimental results for 4 specimens has been performed.
2. The moment carrying capacity of short span is higher than longer span. But some specimen with larger in overall depth like P1 & P2 has higher moment carrying capacity even though the span is comparatively higher.
3. From the comparison of failure modes and moment carrying capacity of experimental and theoretical data, it was observed that there were showing varying in results between them for most of the sections. Therefore, further refinement has to be done for theoretical calculation as part of this study.

6. CONCLUSION

In this thesis, the moment carrying capacity and failure modes of Latticed built-up I beam section has been analysed both theoretically and experimentally and the results also been compared between them.

1. Different modes of failure and ultimate moment for different sections has been obtained.
2. From the comparison of failure modes and moment carrying capacity of experimental and theoretical data, it was observed the theoretical results are not in correlation with experimental results. Therefore, further refinement has to be done for theoretical calculation as part of this study.
3. From the evaluation of experimental results, it shows that the specimen P2- 50-50-2-15-300-2400 has better load carrying capacity compared to other specimens due to its comparatively higher in depth of section and also due to comparatively higher thickness of CFS sheet used in fabrication. Moment carrying capacity of such type latticed beams depends on the spacing of lacing and the pattern of bending of lacing.

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