Revista ALCONPAT
www.revistaalconpat.org
eISSN 2007-6835
Revista de la Asociación Latinoamericana de Control de Calidad, Patología y Recuperación de la Construcción

Extension of direct strength method to two dimensional cold formed steel frame

P. W. Kubde[1*], K. K. Sangle[1]
*Contact Author: pwkubde@vjti.org.in
DOI: http://dx.doi.org/10.21041/ra.v7i1.196

Received: 11/12/2016 | Accepted: 27/04/2017 | Published: 31/05/2017

ABSTRACT
The use of cold-formed steel (CFS) structures is increasing due to the advances in manufacturing, construction technologies and relevant standards. CFS has many advantages. However, the design of CFS structures is complex because of their thin walled open sections making them vulnerable to torsional-flexural buckling and local buckling. Direct strength method (DSM) is the method available for individual beam and column, with certain limitations. To overcome one such limitation this paper attempts to find a formula as an extension to DSM. Already experimented frame was used to validate software model and same frame with different heights was analysed by Finite Element Method and DSM and a formula is obtained as an extension to DSM.
Keywords: cold formed steel; distortional buckling; global buckling; direct strength method; finite element analysis.

Citation: P. W. Kubde, K. K. Sangle (2017). "*Extension of direct strength method to two dimensional cold formed steel frame*", Revista ALCONPAT, 7 (2), pp. 172-185, DOI: http://dx.doi.org/10.21041/ra.v7i1.196

[1]Structural Engineering Department, Veermata Jijabai Technological Institute (VJTI), Mumbai, 400019, India.

Legal Information
Revista ALCONPAT is a quarterly publication of the Latinamerican Association of quality control, pathology and recovery of construction- International, A.C.; Km. 6, Antigua carretera a Progreso, Mérida, Yucatán, México, C.P. 97310, Tel.5219997385893. E-mail: revistaalconpat@gmail.com, Website: www.revistaalconpat.org.
Editor: Dr. Pedro Castro Borges. Reservation of rights to exclusive use No.04-2013-011717330300-203, eISSN 2007-6835, both awarded by the National Institute of Copyright. Responsible for the latest update on this number, ALCONPAT Informatics Unit, Eng. Elizabeth Maldonado Sabido, Km. 6, Antigua carretera a Progreso, Mérida Yucatán, México, C.P. 97310.
The views expressed by the authors do not necessarily reflect the views of the publisher.

Any discussion, including authors reply, will be published on the first number of 2018 if received before closing the third number of 2017.

Ampliação do método da resistência direta para dimensionar paredes de steel frame

RESUMO

O emprego de paredes de steel frame (CFS) está crescendo devido à evolução da tecnologia de manufatura, ao desenvolvimento da construção civil e devido a existência de normas técnicas, conferindo vantagens ao seu emprego. Entretanto o projeto de estruturas de CFS é complexo devido a pouca espessura das paredes e a existência de aberturas (caixilhos e portas) deixarem essas estruturas sujeitas a falhas por flambagem localizadas, ocasionadas por flexão combinada com torsão. O chamado Método da Resistência Direta, DSM, é o método disponível, com certas limitações, para dimensionar vigas e pilares. Para superar essas limitações este artigo se dedica a propor uma fórmula como ampliação e melhoria do método tradicional de dimensionar por DSM. Uma estrutura de steel frame foi analisada experimentalmente para validar o modelo empregado no programa de cálculo (software) e a mesma estrutura, com diferentes alturas, foi analisada pelo método DSM e análise por elementos finitos, com vistas a obter uma fórmula que ampliou e melhorou o espectro de uso do DSM.

Palavras-chave: steel frame; análise de elementos finitos; método da resistência direta DSM; flabagem por torsão; flambagem generalizada.

Ampliação do método da resistência direta para dimensionar paredes de steel fram

RESUMEN

O emprego de paredes de steel frame (CFS) está crescendo devido à evolução da tecnologia de manufatura, ao desenvolvimento da construção civil e devido a existência de normas técnicas, conferindo vantagens ao seu emprego. Entretanto o projeto de estruturas de CFS é complexo devido a pouca espessura das paredes e a existência de aberturas (caixilhos e portas) deixarem essas estruturas sujeitas a falhas por flambagem localizadas, ocasionadas por flexão combinada com torsão. O chamado Método da Resistência Direta, DSM, é o método disponível, com certas limitações, para dimensionar vigas e pilares. Para superar essas limitações este artigo se dedica a propor uma fórmula como ampliação e melhoria do método tradicional de dimensionar por DSM. Uma estrutura de steel frame foi analisada experimentalmente para validar o modelo empregado no programa de cálculo (software) e a mesma estrutura, com diferentes alturas, foi analisada pelo método DSM e análise por elementos finitos, com vistas a obter uma fórmula que ampliou e melhorou o espectro de uso do DSM.

Palabras clave: steel frame; análise de elementos finitos; método da resistência direta DSM; flabagem por torsão; flambagem generalizada.

1. INTRODUCTION

In India, due to continuously increasing industrialization as well as heavy population, cold-formed steel structures like industrial storage rack structures and mass housing are the need of the hour, for which cold-formed steel frames can prove as very economical and efficient alternative. Cold-formed steel has advantages of attractive appearance, fast construction, low maintenance, easy extension, lower long-term cost, non-shrinking and non-creeping at ambient temperatures, no requirement of formwork, termite-proof and rot proof nature, uniform quality, non combustibility.

1.1 Direct strength method-A brief

There are two basic design methods for CFS members viz, The Traditional Effective Width method (Yu W. W., 2000) and upcoming, continuously developing but relatively less known, DSM which was adopted in the North American Design specifications in 2004 as an alternative to Effective Width Method. Appendix 1 of the North American Specification for the design of CFS structural Members, 2004, supplement to the 2001 edition talks about formulae and applications of DSM for columns and Beams. CFS members, due to their thin walls, put before engineers challenges like local plate buckling and cross section distortion. But the same challenge provides the advantage like local plate buckling has the capacity for post buckling reserve strength, which makes the member efficient. DSM is one such method which answers the above challenges at the same time uses the opportunity created by the challenges. (Schafer B. W., 2006).

The development of the DSM started at the University of Sydney by research in to distortional buckling of rack post sections (columns) and was further developed for method of beams. (Schafer B. W., 2006). Further Hancock (Hancock et. al, 1994) showed that compressive strength in a distortional failure correlated well with the slenderness in the elastic distortional mode.

Accurate member elastic stability is the fundamental to the DSM. The method is based on the idea that if all the three elastic instabilities viz, local, distortional and global buckling, along with load or moment causing yielding of the section can be found out, then the strength can be directly determined. The method uses column curves for global buckling with application to local and distortional buckling instabilities. The increased accuracy of the method occurs due to improvements in the local buckling prediction. The method also takes into consideration, deflection calculation (Serviceability). It is a reliable method; its reliability being established using limit state design format in use in the United States. (Schafer B. W., 2006)

Appendix 1 of the North American Specification for the design of CFS structural Members, 2004, supplement to the 2001also includes a number of tables that provide the geometrical and material bounds members which passed in the verification of the direct strength approach, in the process of codification of the same. Pre-qualified is the apt name that has been given to theses sections. Although this method is mainly due to Schafer B. W. (Schafer B. W., 2000), Moen C. along with Schafer B. W., took this method further by developing DSM equations that are applicable to CFS structural members with perforations (Moen C. D. and Schafer B. W., 2006; 2008; 2010; 2011) In 2006 Schafer B. W. came out with a guide “Direct Strength Design method guide”. (Schafer B. W., 2006).

1.2 Literature review

A study on the flexural strength and deflections of discretely braced cold-formed steel C and Z sections was conducted at the University of Florida (Ellifrit D., 1991; Ellifrit D., 1992; Ellifrit D., 1997). In the research, typical C and Z sections were tested in flexure with various types of bracing. Researchers developed a finite element model for the nonlinear large-deflections and rotation analysis of beam-columns. (Pi YL, 1994; Pi YL, 1994). Researchers Performed lateral buckling tests on unbraced, simply supported cold-formed lipped channel beams. A vertical load was applied at the shear centre of the section, or at a point below the shear centre. The beams were supported at the ends by connecting them to a steel block with two bolts at the web of the section (Bogdan M. Put, 1999; 1999).

In another study, researchers (Yu C., 2003; Schafer B. W., 2006) studied the buckling behaviour of CFS channel beams. The buckling test was carried out on simply supported unbraced CFSs of two different cross sections. The lateral buckling test results showed that the CFS sections failed catastrophically by local & distortional buckling of most compressed elements of the cross section after quite large deformations. The results of 10 lateral buckling tests on simply supported CFCs of two different cross sections were presented & also compared with analytical design method as per AS 4100 (AS 4100 Australian Standard Steel Structures ,1981). It was found that the moments at failure were lower when the beam lateral defections increased the compression in the compression flange lip, and higher when they increased the compression in the flange-web junction. Researchers (Put B. M. et al., 1998). performed a local buckling test on CFS C-sections and Z-sections During the test, bracing had been carefully considered in these tests to insure that distortional buckling and lateral-torsional buckling do not influence the interpretation of results. They concluded that the test results can be used for the evaluation of existing and proposed methods for strength prediction of webs in local buckling. In addition, these tests can form the basis for later evaluations in which restrictions on the distortional mode are relieved.

Overall test results indicate that AISI (American Iron and Steel Institute, 1996) design method provides adequate strength predictions. The DSM provides the best test-to-predicted ratio for both slender and 'unslender' specimens. The test results demonstrate that many improvements in the elastic buckling and effective width calculation of C's and Z's are still possible.

Researchers studied the flexibility of beam-to-column connectors used in thin walled cold-formed steel pallet racking systems (American Iron and Steel Institute,1996) The attention is focused on beams subject to torque, because of the effect of transverse loads not applied at the shear centre (Bajoria, K.M. et al., 2006). A simple geometric nonlinear analysis method, based on satisfying equilibrium in the deformed configurations, is examined and used to predict the behavior of the beams. Simple geometric analyses, finite element analyses and finite strip analyses are performed and compared with experimental results. The influence of typical support conditions is studied and they are found to produce a partial warping restraint at the ends. This effect is accounted for by introducing a hypothetical spring. The magnitude of the spring stiffness is assessed for commonly used connections. Other factors that affect the behaviour of cold-formed steel members, such as local buckling, are also studied. The goal was to solve for the unknown rotation. The tested results were comparing with available standard commercial finite element software. The results matched closely, for the problems of interest, up until yielding takes place in the member. This can be seen in the results provided in the later sections. Therefore, it can be concluded that, for the problems of interest, the major contribution to nonlinearity, in the elastic range, is the nonlinearity due to the dependence of the torque on the rotation of the beam. They concluded that, under load, the beam displaces horizontally and rotates gradually, but no sudden lateral buckling of the beam takes place. The failure is started by yielding of the material. The AISI procedure for the estimation of the strength, based on lateral-torsional buckling, may under- or overestimates the strength. The beam undergoes large rotation before failure. Therefore, the serviceability limit state may be more critical than the strength limit state. In the case of beams with no slender elements, FEA by modelling as beam elements or shell elements predict similar behaviour.

A new CFS section was manufactured and tested using newly proposed double cantilever method. Bajoria and Talicotti (Bajoria, K. M., 2006) proposed alternative beam to column test instead of the cantilever test. Complete studies involving both experimental and numerical investigations were performed to find out the flexibility of the beam-to-column connector, and this was followed by a full scale frame test to compare the results obtained. The double cantilever test takes into account the realistic behaviour of connectors, which are subjected to moment, shear and axial pull by the beams. This was confirmed by the results of the full-scale frame test.

Researchers studied the three dimensional (3D) model of conventional pallet racking systems using the finite element program ANSYS. They carried a free vibration modal analysis on conventional pallet racks with the 18 types of column sections developed along with semi-rigid connection. The stiffness of the connector was tested using the conventional cantilever method and also using a double cantilever method. They performed nonlinear FEA of both the tests. The model is aimed at developing simplified equation for the fundamental period of storage racks in their down aisle direction. Finally, parametric study was carried out to find out the fundamental mode shape and time period. Finite element method was used for the accuracy and appropriateness of cold-formed steel frame.

In 2011, K. K. Sangle performed the finite element buckling and dynamic analyses of two-dimensional (2D) single frames and three-dimensional (3D) frames of cold-formed sections with semirigid connections used in the conventional pallet racking system.(Bajoria, K. M. et al.,2011) The results of buckling analysis for the single 2D frames are compared with those from the experimental study and effective length approach given by RMI (RMI Specification for the design, testing and utilization of industrial storage racks, Rack Manufacturers Institute,2008.)
The finite element model used for the single 2D plane frame is further extended to 3D frames with semirigid connections, for which the buckling analysis results are obtained. Researchers. (Thombare C. N. et.al., 2016) studied nonlinear buckling analysis of 2D cold formed steel storage rack structure using appropriate commercial FE platform. In this paper, the FEA results are validated with available experimental data for a particular two dimensional CFS frame (Bajoria, K.M., 2011) with different thicknesses and bracing patterns. The FEA results show good convergence with the experimental results. Hence, study is extended to more heights of the frame. The results are then used to suggest an extension formula for modification in current DSM expression for two dimensional frames.
Significant research is going on to simplify the design of CFS sections and frames to make it more reliable and practically acceptable. The addition of perforations serves lots of advantages for the practical purpose, but at the same time it generates complications in design. Considerable efforts are required to study the impact of holes on the strength of the member. The aim of the current research is to propose a formula which will help to find out load carrying capacity of two dimensional CFS with varying heights as an extension to the DSM equations.
In this paper, the FEA results are validated with available experimental data for a particular two dimensional CFS frames (Bajoria, K.M., 2011), with different thicknesses and bracing patterns. The FEA results show good convergence with the experimental results. Hence, the study is extended to more heights of the frame. The results are then used to suggest an extension formula for modification in current DSM expression for two dimensional frames.

2. RESEARCH DESIGN

The type of research is explorative and analytical. CFS Frames selected are as per availability and hence convenient sampling method is adopted. A sample size of 12 frames each one having different properties have been used for validation of FEA model, as experimental data was available for these frames. (Table1). Same thing applies to all other tables; i.e. in every table every frame is having different properties.

3. ANALYTICAL STUDY

In this study, the finite FEA results are validated with available experimental data for a particular two dimensional CFS frame. (Bajoria, K. M. et.al., 2011) with different thicknesses (shown in figure 3). Same frames were then used for analytical study by two methods viz. DSM and FEM using software; for DSM, software -CUFSM version 4.05 which is freely available was used and for FEM, appropriate commercial software.

3.1 Finite element modelling

Element type: Three elements commonly used by FEM software in the elastic buckling analysis of thin walled structures are the S9R5, S4 and S4R elements as shown in fig.1. The S4 and S4R are four node general purpose shell elements, valid for both thick and thin shell problems. Both elements employ linear shape functions to interpolate deformation between nodes. S4R element has been used for meshing the model. Advantage of S4R element over S4 is, S4R uses reduced integration with hourglass control, finite membrane strains.

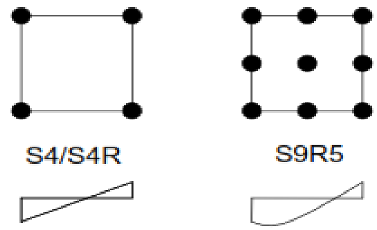


Figure 1. FE S4\S4R shell element and FE S9R5 shell element

Boundary condition and loading: Boundary condition for the modelled column is pinned-pinned, free to wrap. End cross section nodes are restrained in X and Z direction and nodes at the centre are restrained in Y direction to prevent Rigid Body motion. A reference load of 1 kN is applied as a shell edge load over the perimeter of the column. The column has perforations at 140 cm c/c. Modulus of Elasticity is 212000 MPa and Poission's Ratio is 0.325
Analysis: Liner Eigen buckling analysis was performed with appropriate FEA software. Eigen value obtained from the analysis is used to calculate the buckling capacity of the frame. The models were meshed with 30mm and 10mm mesh for convergence study. It was observed that the convergence was non-monotonic, hence to find more accurate eigenvalue, the models with 10mm mesh is used.

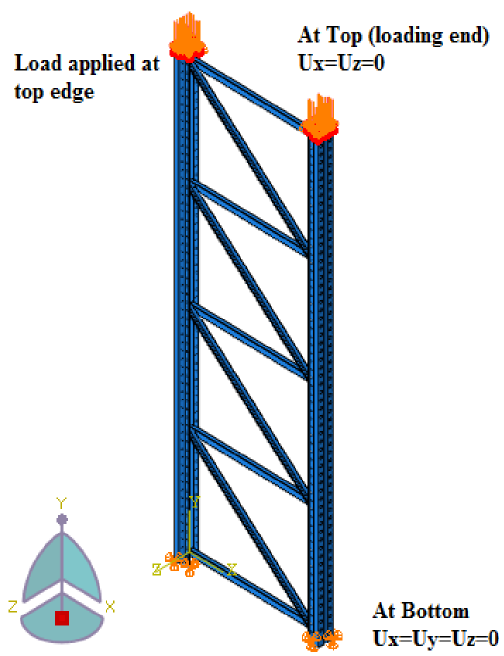


Figure. 2. Boundary Conditions used in FEA model.

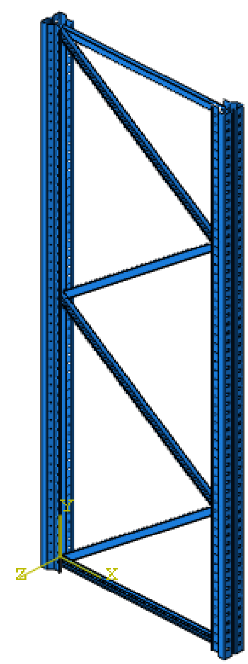


Figure.2 (a). Bracing Type B1 (Inclined bracing only).

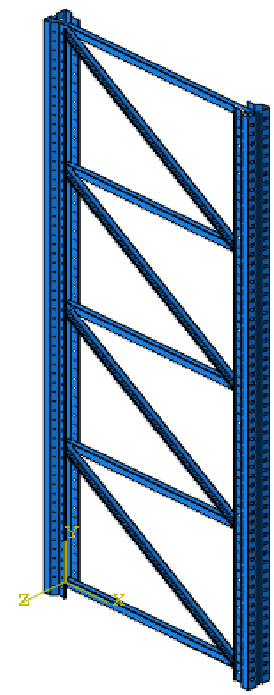
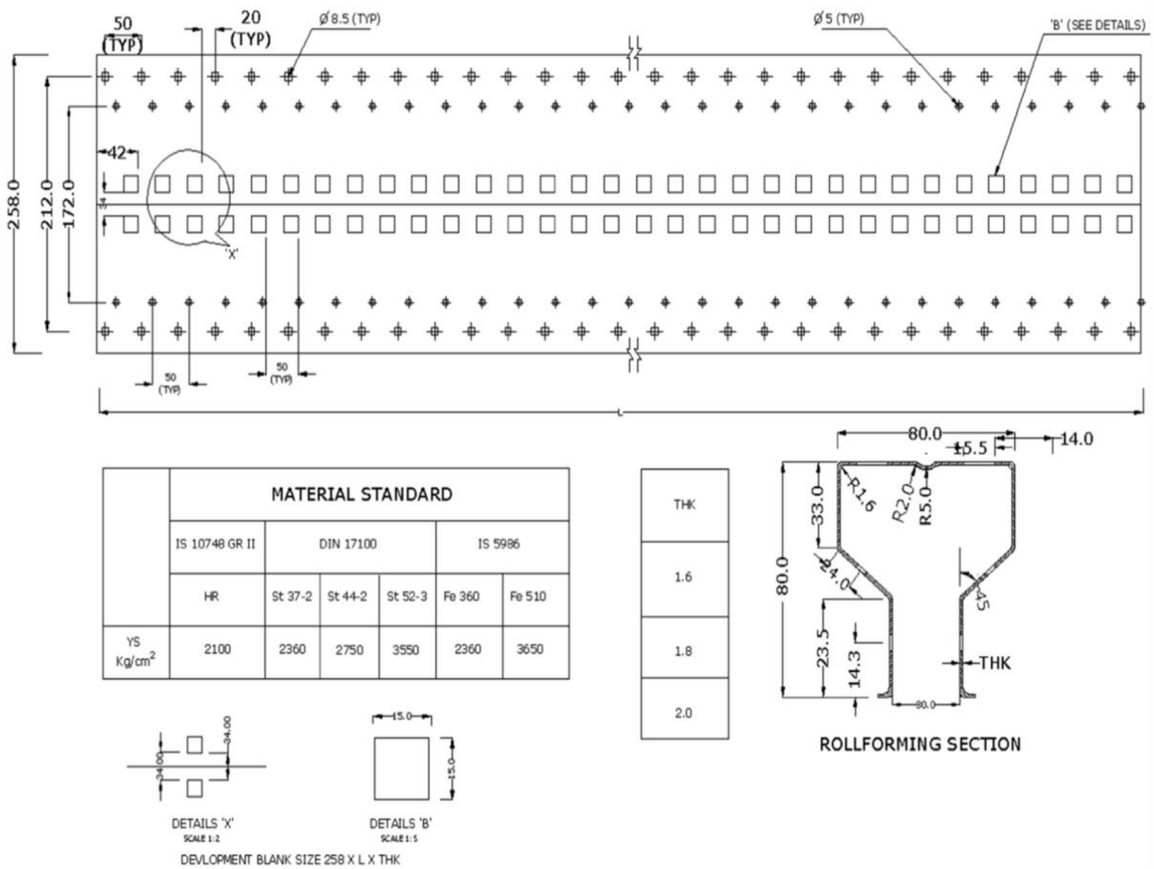


Figure.2 (b). Bracing Type B2 (Horizontal and inclined bracing).

3.2 Extending DSM for frame
In this section, an attempt was made to solve an axially loaded frame using equations given by (Schafer B.W., Moen C.D., 2010) and corresponding modifications was given. A frame is modelled in CUFSM v4.05 as shown in fig 4. The bracing effect was not considered as CUFSM is applicable only for compression and flexural member analysis.



ALL DIMENSIONS ARE IN MM.

Figure 3. Details of Column cross section and Column forming the frame. (Bajoria, K. M. et. al., 2011)

The following material properties were assigned to the section.
Yield Stress = 365 N/mm^2
Young's modulus = E = 212000 N/mm^2
Shear Modulus = G= 80000 N/mm^2
Poison's ratio = v = 0.325
Thickness = 1.6 mm throughout the section.
Two column sections were connected by elements with zero thickness as shown in figure 4. The cross sectional properties are found by the 'section properties option' of the software as shown. The load applied is 1000 N on the cross section and to generate a frame effect, connecting zero elements were deleted as shown (Figure 4). Similarly for net cross sectional properties, zero element thickness were assigned to hole region. Again load of 1000 N was applied and zero elements were deleted as shown. Before analysis the net cross section of frame, the corner nodes of member were restrained in Z-direction (Schafer B.W., Moen C.D., 2010). The column section of frame is having 2 no's of holes on the web and 4 no's of holes on flange. Thus, while the calculations of perforations in frame total 4no.s of holes were assumed in the evaluation of strength.

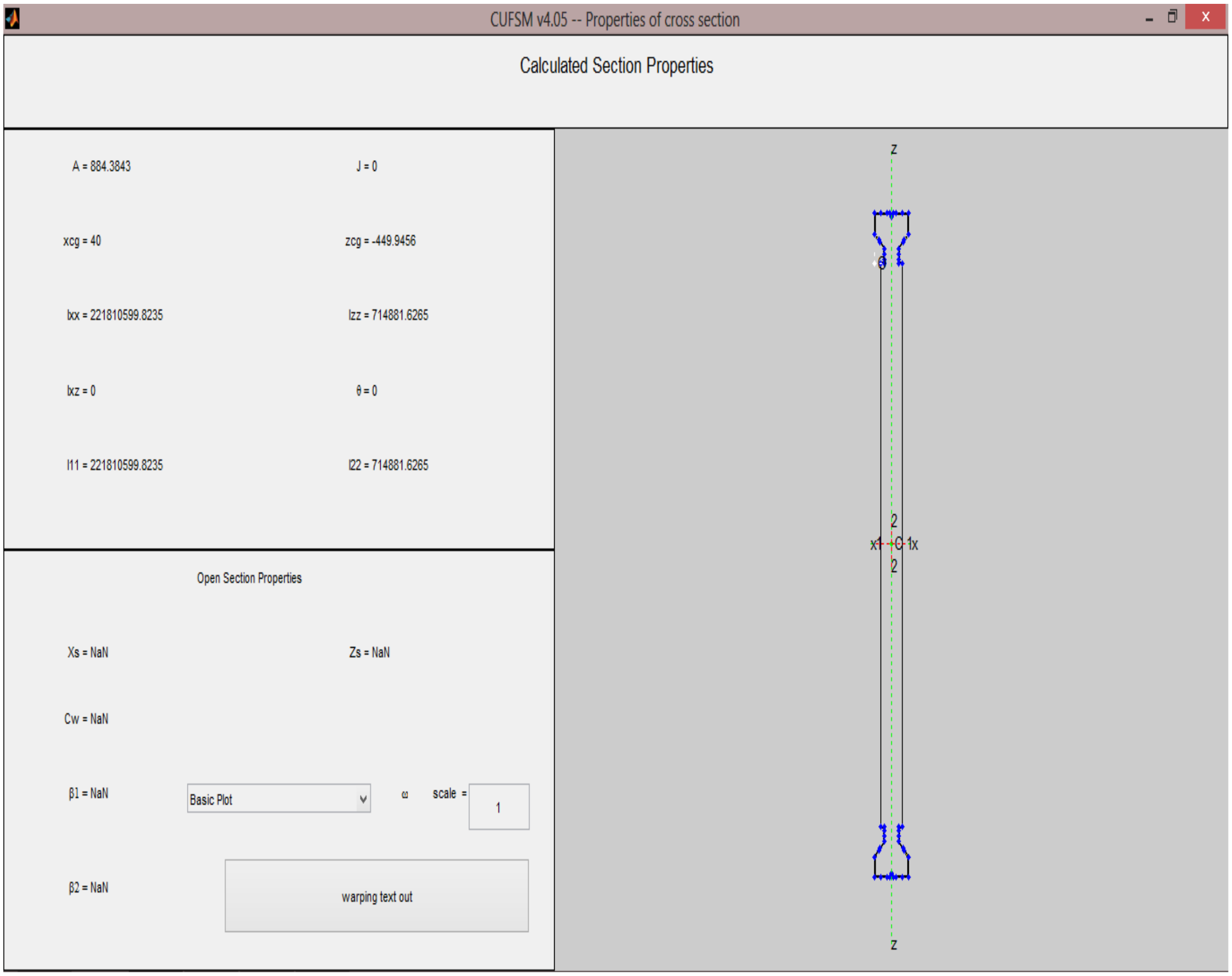


Figure 4. Finite strip model of frame MW1.6 in CUFSM version 4.05

4. RESULTS

In table 1, experimental results already obtained by earlier researchers for various frames and FEM results obtained for the same frames are tabulated.
{Meaning of Nomenclature in Column 1of table 1: For example: MW-1.6-B1 is Column frame Name: where MW means medium weight (HW means heavy weight), 1.6mm is thickness , B1 is the bracing type as shown in figure 2a ,(B2 is other bracing type as shown in figure 2b.)}

Table 1. Validation of model: Experimental and FEA results .

Experimental study 1: without spacer bars (3.1 m height frame)				
column frame	**Pe (experimental) kN**	**Pe (FEM) kN**	**% error**	**Pe (FEM) / Pe (experimental)**
MW-1.6-B1	103.51	112.04	-8.24	1.08
MW-1.6-B2	115.45	125.00	-8.27	1.08
MW-1.8-B1	166.78	151.15	9.37	0.91
MW-1.8-B2	176.88	160.74	9.12	0.91
MW-2.0-B1	200.41	182.29	9.04	0.91
MW-2.0-B2	215.46	196.81	8.66	0.91

HW-2.0-B1	223.45	236.20	-5.71	1.06
HW-2.0-B2	235.26	269.00	-14.34	1.14
HW-2.25-B1	264.24	268.65	-1.67	1.02
HW-2.25-B2	275.56	304.40	-10.47	1.10
HW-2.5-B1	295.46	301.63	-2.09	1.02
HW-2.5-B2	305.56	340.12	-11.31	1.11
Average Ratio				1.11
Coefficient Of Variation.				11.08

In the following tables 2, 3 and 4, FEM and DSM results for the same model validated in table 1 but with three different heights. (Table 2: height 3.1 m, Table 3: Height 4.6m and Table 4: height 6.1m) are presented. As the FEM model is already validated, the FEM values for frames of other heights with holes are considered to be correct and are compared with corresponding DSM values with holes to find out required appropriate formula / factor which when used along with DSM value will give correct value i.e. same value as is given by FEM.

Table 2. Critical Buckling Strength for frames of height 3.1m

3.1 m frame height						
column frame	**Pe (FEM) kN**	**Pe (DSM) kN**		**% error**		$\frac{\text{DSM with hole}}{\text{FEM}}$
		without Hole	**With Hole**	**without Hole**	**With Hole**	
MW-1.6-B1	112.04	169.92	94.71	-64.16	15.46	0.85
MW-1.8-B1	151.15	191.16	106.55	-14.62	29.51	0.70
MW-2.0-B1	182.29	212.40	118.39	-5.98	35.06	0.65
HW-2.0-B1	236.20	253.99	228.32	-13.67	3.34	0.97
HW-2.25-B1	268.65	291.89	262.89	-10.46	2.14	0.98
Average Ratio						0.83
Coefficient Of Variation.						-16.33

Table 3. Critical Buckling Strength for frames of height 4.6m

4.6 m frame height						
column frame	**Pe (FEM) kN**	**Pe (DSM) kN**		**% error**		$\frac{\text{Pe(DSM) with hole}}{\text{Pe(FEM)}}$
		without Hole	**With Hole**	**without Hole**	**With Hole**	
MW-1.6-B1	73.20	169.92	42.98	-132.13	41.29	0.59
MW-1.8-B1	82.39	191.16	48.35	-132.01	41.32	0.59
MW-2.0-B1	91.71	212.40	53.72	-131.60	41.42	0.59
HW-2.0-B1	154.08	253.99	122.42	-64.85	20.54	0.79
HW-2.25-B1	172.16	291.89	141.21	-69.54	17.98	0.82
HW-2.5-B1	190.65	317.49	153.03	-66.53	19.74	0.80
Average Ratio						0.70
Coefficient Of Variation.						15.36

Table 4. Critical Buckling Strength for frames of height 6.2m

6.2 m frame height						
Column frame	Pe (FEM) kN	Pe (DSM) kN		% error		$\frac{\text{DSM with hole}}{\text{FEM}}$
		without Hole	With Hole	without Hole	With Hole	
MW-1.6-B1	53.89	169.92	23.71	-215.30	56.01	0.44
MW-1.8-B1	60.31	191.16	26.67	-216.95	55.78	0.44
MW-2.0-B1	66.73	212.40	29.63	-218.31	55.59	0.44
HW-2.0-B1	123.31	253.99	67.56	-105.97	45.21	0.55
HW-2.25-B1	136.93	291.89	77.93	-113.16	43.09	0.57
HW-2.5-B1	150.70	317.49	84.45	-110.67	43.96	0.56
Average Ratio						0.50
Coefficient Of Variation.						12.06

From table 2, 3 and 4, it is observed that ratio of DSM with hole to FEM shows less variation for same height of the frame. The average of these ratios, say R has been plotted against the height of corresponding frame (say R) (Refer Figure 5).
From above graph presented in figure 5, we get the following equation for frame with bracing type B1,

$$R = -0.1241 \text{ x Frame Height} + 1.2694 \qquad (1)$$

Where R is the Avg. ratio factor i.e. ratio of strength by FEM to the strength by DSM with hole. Using this ratio and strength of frame calculated by DSM with hole, we can find out the critical buckling strength for any height of the frame with bracing type B1.
Table 5 represents FEM values and values from DSM with modification factor for frames with bracing type B1, which makes it clear that factor R obtained from equation (1) when applied to DSM value, modifies that value bringing it close to the FEM value.

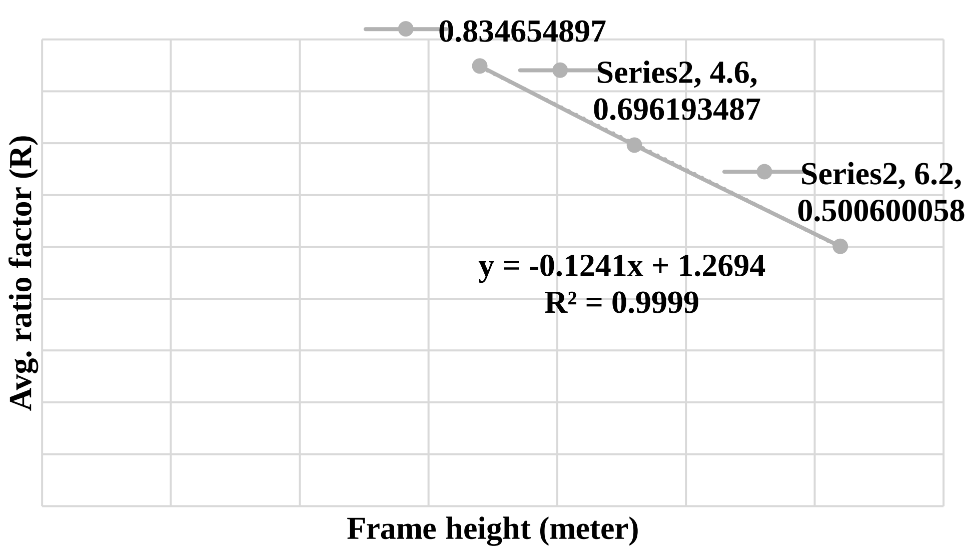


Figure 5: Frame Height vs. Avg. Ratio Factor

Table 5. Comparison of DSM results with FEM for frame with bracing B1

Frame Height (m)	Frame type	Strength using FEM for B1 type frame (kN)	Strength using DSM equations (kN)	Strength by developed equation (kN)	FEM / Strength by DSM equation	FEM / Strength by developed equation
3.1	MW-1.6	112.04	94.711	107.06	0.85	1.05
3.1	MW-1.8	151.15	106.55	120.44	0.70	1.26
3.1	MW-2.0	182.29	118.389	133.82	0.65	1.36
3.1	HW-2.0	236.20	228.318	258.08	0.97	0.92
3.1	HW-2.25	268.65	262.892	297.16	0.98	0.90
3.1	HW-2.5	301.63	285.397	322.60	0.95	0.94
4.6	MW-1.6	73.20	42.978	61.53	0.59	1.19
4.6	MW-1.8	82.39	48.351	69.22	0.59	1.19
4.6	MW-2.0	91.71	53.723	76.91	0.59	1.19
4.6	HW-2.0	154.08	122.421	175.25	0.79	0.88
4.6	HW-2.25	172.16	141.207	202.15	0.82	0.85
4.6	HW-2.5	190.65	153.027	219.07	0.80	0.87
6.2	MW-1.6	53.89	23.707	47.42	0.44	1.14
6.2	MW-1.8	60.31	26.67	53.34	0.44	1.13
6.2	MW-2.0	66.73	29.634	59.27	0.44	1.13
6.2	HW-2.0	123.31	67.563	135.13	0.55	0.91
6.2	HW-2.25	136.93	77.93	155.87	0.57	0.88
6.2	HW-2.5	150.70	84.453	168.91	0.56	0.89
Average Ratio						1.04
Coefficient Of Variation.						15.15

Table 6 represents FEM values and values from DSM with multiplication factor for frames with bracing type B2 (for which results and analysis are not presented in this paper) , which makes it clear that multiplication factor obtained from equation for bracing type B2 when applied to DSM value, modifies that value bringing it close to the FEM value.

Table 6. Comparison of DSM results with FEM for frame with bracing B2

Frame Height	Frame type	Strength using FEM for B2 type frame	Strength using DSM equations (kN)	Strength by developed equation (kN)	FEM / Strength by DSM Equations	FEM / Strength by developed equation
3.1	MW-1.6	125.00	94.711	126.33	0.76	0.99
3.1	MW-1.8	160.74	106.55	142.12	0.66	1.13
3.1	MW-2.0	196.81	118.389	157.91	0.60	1.25
3.1	HW-2.0	269.00	228.318	304.53	0.85	0.88
3.1	HW-2.25	304.40	262.892	350.65	0.86	0.87
3.1	HW-2.5	340.12	285.397	380.66	0.84	0.89
4.6	MW-1.6	88.66	42.978	72.60	0.48	1.22
4.6	MW-1.8	99.56	48.351	81.68	0.49	1.22
4.6	MW-2.0	110.38	53.723	90.75	0.49	1.22
4.6	HW-2.0	194.04	122.421	206.80	0.63	0.94

4.6	HW-2.25	236.21	141.207	238.53	0.60	0.99
4.6	HW-2.5	215.21	153.027	258.50	0.71	0.83
6.2	MW-1.6	61.94	23.707	55.95	0.38	1.11
6.2	MW-1.8	69.45	26.67	62.94	0.38	1.10
6.2	MW-2.0	76.92	29.634	69.94	0.39	1.10
6.2	HW-2.0	154.61	67.563	159.46	0.44	0.97
6.2	HW-2.25	171.96	77.93	183.92	0.45	0.93
6.2	HW-2.5	189.13	84.453	199.32	0.45	0.95
Average Ratios						0.13
Coefficient Of Variation.						12.94

By using the respective multiplication factors R , the buckling strength for few more heights has been calculated and shown in table 7, for both type of bracings B1 and B2{Unlike frame type B1(Tables 2 to5) , detailed results and analysis for frame type B2 are not presented in this paper}.

Table 7. Buckling Strength of frames for few more heights.

Frame Type	**Frame Height (m)**	**Strength using DSM equations (kN)**	**Strength by developed equation for frame (kN)**	
			with Bracing type B1	**with Bracing type B2**
MW-1.6	3.8	63	78.97	93.18
MW-1.8	3.8	70.87	88.83	104.82
MW-2.0	3.8	78.75	98.71	116.47
HW-2.0	3.8	175.27	219.69	259.23
HW-2.25	3.8	202.01	253.20	298.78
HW-2.5	3.8	219.09	274.61	324.04
MW-1.6	5.3	32.36	52.90	62.43
MW-1.8	5.3	36.41	59.53	70.24
MW-2.0	5.3	40.46	66.15	78.05
HW-2.0	5.3	92.19	150.72	177.85
HW-2.25	5.3	106.34	173.85	205.15
HW-2.5	5.3	115.24	188.40	222.31
MW-1.6	6.8	19.65	46.18	54.49
MW-1.8	6.8	22.11	51.96	61.31
MW-2.0	6.8	24.57	57.74	68.13
HW-2.0	6.8	55.98	131.56	155.24
HW-2.25	6.8	64.57	151.74	179.06
HW-2.5	6.8	69.87	164.20	193.75

5. CONCLUSION

1. Main objective of the research was to verify whether the DSM developed by Moen is applicable to two dimensional CFS frames or not, and if not, then to suggest appropriate modification in existing DSM. Hence two dimensional CFS frames studied by K. M. Bajoria, K. K. Sangle and R. S. Talicotti, 2011, at IITB Mumbai were used for validation of FEA model . Load carrying capacities of the same model with same height as well as

with few different heights were then calculated using FEM software, as well as DSM formulae (software Cufsum). All these FEM and DSM values tabulated in tables 2, 3, 4 were then graphically analysed (Figure 5) to obtain a multiplication factor R equation (1). This multiplication factor, researched in this paper, which when applied to DSM value obtained for the frame of any height, gives correct load carrying capacity (FEM value) of that frame with an accuracy of +/- 20%. (Table 5 and 6).

2. Applying the correction factor found in this paper to DSM formulae, load carrying capacity for the frame of any height can be calculated instead of carrying out rigorous and tedious FEA.

5. REFERENCES

Research Papers from Journals

Bajoria, K. M., Talikoti, R. S. (2006), "*Determination of fexibility of beam-to-column connectors used in thin walled cold-formed steel pallet racking systems*", Journal of Thin-Walled Structures, Vol. 44, Pages 372-380.

Bajoria, K. M. Sangle K. K., Talikoti, R. S. (2011), "*Stability and Dynamic Analysis of Cold-Formed Storage Rack Structures with Semirigid Connections*", Journal of Thin-Walled Structures, Vol. 44, Pages 372-380.

Bogdan, M. Put, Yong-Lin Pi, N. S. Trahair, Member, ASCE (1999). "*Lateral Buckling Tests on Cold-Formed Channel Beams*" J. Struct. Eng. 1999.125:532-539.

Bogdan M. Put, Yong-Lin Pi, N. S. Trahair, Member, ASCE (1999). "*Bending and Torsion of Cold-Formed Channel Beams",* J. Struct. Eng.125:504-546; 1999.

Ellifrit D., Sputo T., Haynes J. (1991), *Flexural strength and deflections of discretely braced cold formed steel channel and zee sections*. Project report, American Iron and Steel Institute.

Ellifrit D. et al. (1992), *Flexural capacity of discretely braced C's and Z's. In: Yu W-W*, LaBoube RA, editors. Proceedings of the Eleventh International Specialty Conference on Cold-Formed Steel Structures. St Louis, MO: Department of Civil Engineering, University of Missouri — Rolla, 108–29.

Ellifrit D., Glover B., Hren J. (1997), *Distortional buckling of channels and zees not attached to sheathing*. Report for the American Iron and Steel Institute.

Moen, C. D. (2008), "*Direct Strength Design for Cold-Formed Steel Members with Perforations*". Ph.D. Thesis, Johns Hopkins University, Baltimore.

Moen, C. D., Schafer B. W. (2011), "*Direct strength design method for design of cold formed column with holes*", Journal of Structural Engineering, Vol. 137, No. 5, May 1, 2011 ©ASCE.

Pi, Y. L., Trahair N. S. (1994), *Nonlinear inelastic analysis of steel beam-columns. I: Theory*. Journal of Structural Engineering, 120(7):2041–61; ASCE.

Pi, Y. L., Trahair N. S. (1994) *Nonlinear inelastic analysis of steel beam-columns. II: Applications*. Journal of Structural Engineering, ASC; 120(7):2062–85; ASCE.

Put B. M., Pi Y-L, Trahair N. S. (1998) *Lateral buckling tests on cold-formed channel beams*. Research report no. R767, Center for Advanced Structural Engineering, Department of Civil Engineering, the University of Sydney, Australia.

Schafer, B. W. (2006) "*Direct Strength Method Design Guide*", American Iron and Steel Institute, Washington, D.C. 171 pages.

Schafer B. W., Moen C. D. (2010), "*Extending Direct Design to Cold Formed Steel Beams with Holes*". Twentieth International Specialty Conference on Cold-Formed Steel Structures St. Louis, Missouri, USA.

Schafer B. W., (2006) "*Review: The Direct Strength Method of Cold Formed Steel Member*" Stability and Ductility of Steel Structures. Lisbon, Portugal.

Thombare C. N., Sangle K. K. And Mohitkar V. M. (2016) *Nonlinear buckling analysis of 2D cold formed storage simple cross aisle frames*. Journal of building Engineering, Vol 7(2016), 12-22.
Yu C., Schafer B. W. (2003), *Local buckling tests on cold-formed steel beams*. Journal of Structural Engineering; 129(12):1596–606.

Books

Yu, W.W. Cold-Formed Steel Design. John Wiley & Sons, Inc; 2000.

Standards

American Iron and Steel Institute (1996,. AISI Specification for the Design of Cold-Formed Steel Structural Members. American Iron and Steel Institute. Washington, D.C.;
AS 4100 Australian Standard Steel Structures, (Standards Association of Australia).
RMI, Specification for the design, testing and utilization of industrial storage racks, Rack Manufacturers Institute, 2008.
North American Specification (2004), "*Appendix 1: Design of Cold-Formed Steel Structural Members Using the Direct Strength Method*." Supplement to the North American Specification for the Design of Cold-Formed Steel Structures. American Iron and Steel Institute, Washington, DC; 2004.

Softwares.

Appropriate commercial software for FEM.
CUFSM version 4.05 – A general purpose cold formed steel component design software based on North American Standards