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Nonlinear buckling analysis of 2-D cold-formed steel simple cross-aisle storage rack frames

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ABSTRACT

Industrial storage racks are among the most important structures made from cold-formed steel sections. They are widely used due to the increasing need for rational space utilization in warehouses, and other facilities used to store goods. Pallet rack is a material handling, storage aid system designed to store materials on pallets. Although there are many varieties of pallet racking, all types allow for the storage of palletized materials in horizontal rows with multiple levels. Rack systems are widely used in warehouses where they are loaded with valuable goods. The cold-formed steel columns usually have open cross-sections and are thin walled, making them vulnerable to torsional-flexural buckling and local buckling. The loss of goods may be greater than the total cost of the rack on which the goods are stored, which can indirectly affect the owner. Therefore, understanding the stability of rack structures is very important. This paper deals with numerical linear and nonlinear buckling analysis of 2-D cold-formed steel simple cross-aisle storage rack frames. The main focus of the study is to ascertain the stability of 2-D frames of a pallet racking system. With this objective, a pallet racking system with cold-formed steel sections is simulated by three-dimensional models using shell elements in ABAQUS, a general purpose finite element analysis software. Linear and nonlinear buckling analyses are carried out on these frames. Results are obtained from finite element analysis of frames with 12 types of column sections. Spacer bars and channel stiffeners are used to improve the torsional strength of original open cross sections. Results show that spacer bars and channel stiffeners are very effective in enhancing the strength of cold-formed steel pallet rack structures.

1. Introduction

The performance of 2-D frames of rack structure is very complex due to perforations in column sections and nature of the connections. The performance of storage rack structures depends on how the individual components, like beam column, braces perform uniquely with each other through a designed connection. The analysis and design of thin-walled structures with perforations in open upright cross sections gives many challenges to the structural engineers. Therefore, a thorough understanding of the structural behaviour of rack structures is very important. Presently, only a limited number of design standards, such as the BS EN-15512 [1], Australian code AS4084 [2], AISI [3], SEMA [4] and the specifications published by the RMI [5] provide some guidelines for the analysis and design of rack structures.

Ellifrit et al. [6] studied the flexural strength and deflections of discretely braced cold-formed steel channel and zee sections at the University of Florida. Typical channel and zee sections were tested in flexure with various types of bracing. The load was applied at the junction of web-top flange, i. e. not in the shear centre. Therefore, this is a case of the combined bending and torsion acting on an unbraced beam. However, the effect of torsion was not considered in the analytical modelling. Pi and Trahair [7] developed a finite element model for the nonlinear large-deflections and rotation analysis of beam-columns. Bogdan et al. [8] studied the buckling behaviour of cold formed steel (CFS) channel beams. The buckling test was carried out on simply supported unbraced CFS sections of two different cross sections. The lateral buckling test results showed that the CFS sections failed catastrophically by local and distortional buckling of most compressed elements of the cross section after large deformations. Schafer and Pekoz [9] focused on the performance of the compression flange and did not provide definitive evaluations of the design expressions for the web due to the incomplete restriction of the distortion mode, arrangement of the specimens back to back versus toe to toe, and a

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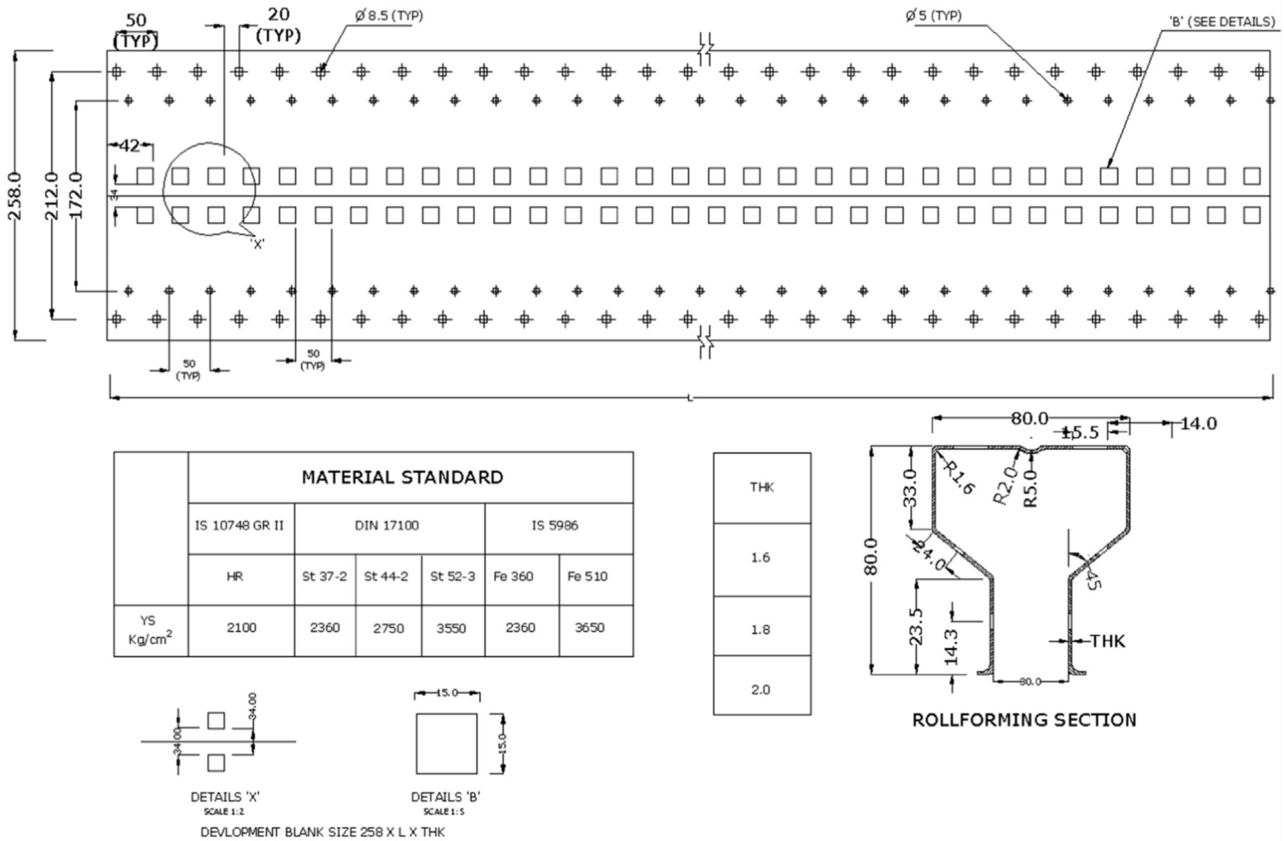
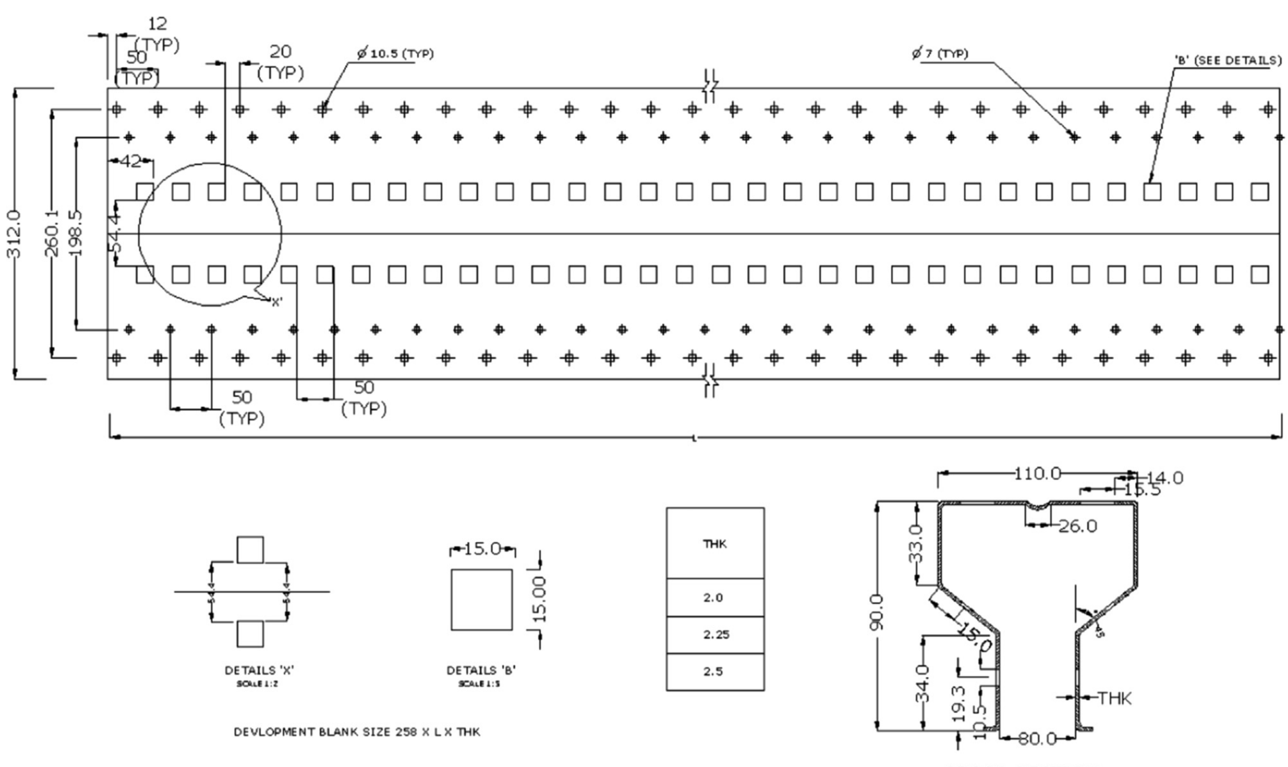


	MATERIAL STANDARD					
	IS 10748 GR II	DIN 17100			IS 5986	
	HR	St 37-2	St 44-2	St 52-3	Fe 360	Fe 510
YS Kg/cm^2	2100	2360	2750	3550	2360	3650

THK
1.6
1.8
2.0

Fig. 1. Medium weight column upright section 1.6 mm, 1.8 mm and 2.0 mm thick.



THK
2.0
2.25
2.5

Fig. 2. Heavy weight column upright section 2.0 mm, 2.25 mm and 2.5 mm thick.

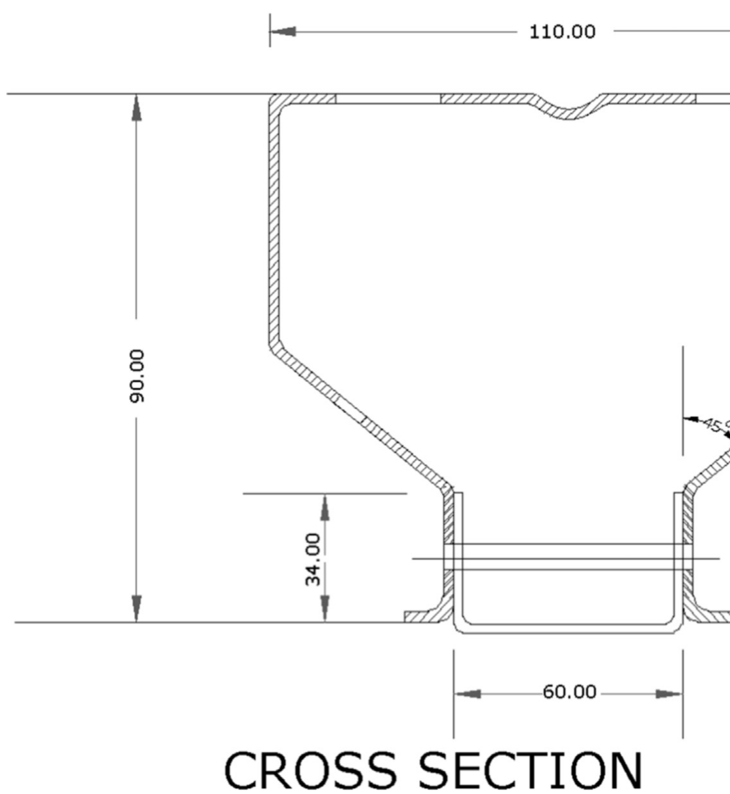


Fig. 3. Torsionally strengthened MW/HW upright section with channel stiffener.

Table 1
Properties of material used in analysis.

Yield stress (MPa)	Ultimate stress (MPa)	Modulus of elasticity E (MPa)	Density (kg/m^3)	Poisson's ratio
365	569	212×10^3	7860	0.29

general lack of information on bracing details. Beale and Godley [10] had performed sway analysis of splice rack structures. The structures evaluated by considering an equivalent free sway column and using computer algebra generated modified stability functions to include the geometric nonlinearity in terms of P-Delta effects. The effect of semi-rigid beam to upright, splice to upright connections were included in the analysis. Each section of upright between successive beam levels in the pallet rack was considered to be a single column element. The results of the analysis were compared with traditional finite element solution of the problem. Godley et al. [11] had performed analysis and design of unbraced pallet rack structures subjected to horizontal and vertical loads. The structures were analyzed by considering an equivalent free-sway column and solving the differential equations of flexure, including P-Delta effects. Initial imperfections within the frame were allowed. The results of the analysis were compared with a traditional non-linear finite element solution of the same problem.

Table 3
Results of the convergence study.

Mesh size of the frame HW-2.0-B1 (height 3.1 m)	50 mm	40 mm	30 mm	20 mm	10 mm	5 mm
Linear Buckling Load (kN)	256.11	242.97	240.70	239.21	**236.2**	236.09

Davies [12,13] worked on the down-aisle stability of rack structures. In their analysis, a single internal upright column carrying both vertical and horizontal loads was used. Freitas et al. [14] worked on analysis of drive-in racks, evaluating the influence of each of their components of global stability. In his study, a full-scale test of a drive-in system was carried out. Finite element models were also developed to evaluate global structural stability and component influence on system behaviour. Baldassino and Bernuzzi [15] worked on the numerical study of the response of pallet racks in Europe. The influence of beam-to-column joint modelling of the overall frame response is singled out with reference to both service condition and ultimate limit states. Schafer [16] studied an 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. Numerical analyses and experiments indicate post buckling

Table 2
Details of the elements used for finite element analysis.

Part of frame	Element name	Description
Column section	S4R	A 4-node doubly curved thin or thick shell, reduced integration, hourglass control, finite membrane strains.
Horizontal bracing	C3D8R	An 8-node linear brick, reduced integration, hourglass control.
Inclined bracing	C3D8R	An 8-node linear brick, reduced integration, hourglass control.
Spacer Bar	C3D8R	An 8-node linear brick, reduced integration, hourglass control.

Table 4
FEA results for frames.

Column frame	P_e Linear in kN (Experimental)	P_e (FEA) kN		% Difference for P_e Linear
		Linear	Nonlinear	
MW-1.6-B1	103.51	116.02	110	−12.09
MW-1.6-B2	115.45	129.52	120	−12.19
MW-1.8-B1	166.78	132.68	130	20.45
MW-1.8-B2	176.88	147.14	140	16.81
MW-2.0-B1	200.41	149.70	145	25.30
MW-2.0-B2	215.46	164.86	160	23.48
HW-2.0-B1	223.45	236.2	230	−5.71
HW-2.0-B2	235.26	269.0	255	−14.34
HW-2.25-B1	264.24	268.65	260	−1.67
HW-2.25-B2	275.56	304.4	300	−10.47
HW-2.5-B1	295.46	301.63	295	−2.09
HW-2.5-B2	305.56	340.12	335	−11.31

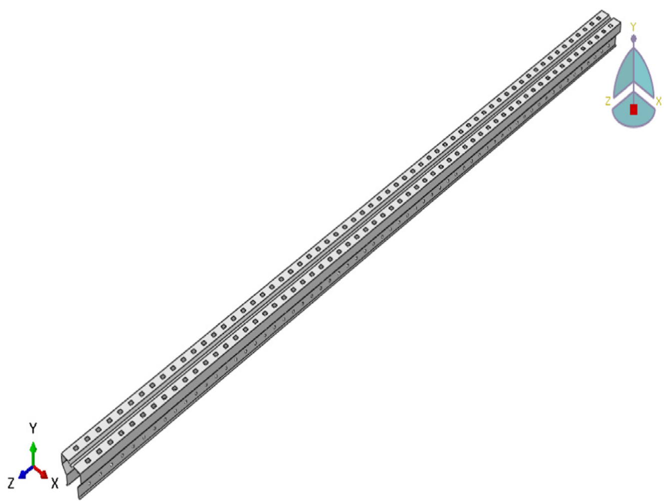


Fig. 4. HW column section modeled in ABAQUS.

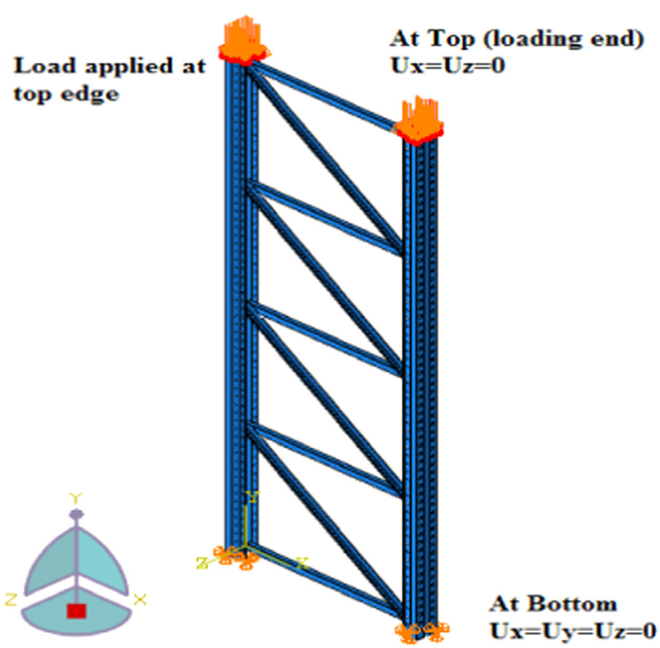


Fig. 5. Boundary conditions used in model.

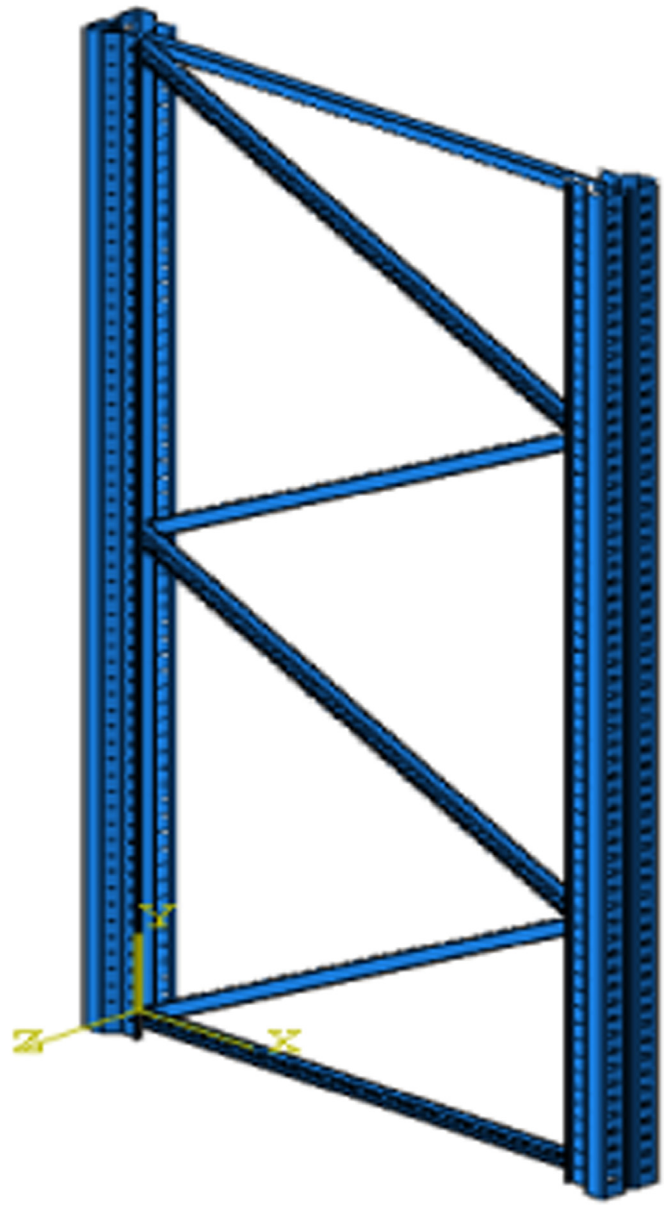


Fig. 6. Bracing Type B1 (inclined bracing only).

capacity in the distortional mode is lower than in the local mode. Bajoria and Talicotti [17] had proposed alternative beam to column test instead of the cantilever test. Their proposed double cantilever test takes into account the actual performance of the connectors, which were subjected to moment, shear and axial pull by the beams. This was validated from the results on full-scale experimental tests. Sangle et al. [18] studied the three dimensional (3D) model of conventional pallet racking systems using the finite element program ANSYS and carried a free vibration modal analysis on conventional pallet racks with 18 types of column sections developed along with semi-rigid connection. Sangle and Bajoria [19] also performed the finite element buckling and dynamic analyses of two-dimensional (2D) single frames and three-dimensional (3D) frames of cold-formed sections with semi rigid connections used in the conventional pallet racking system. The results of buckling analysis for the single 2D frames were compared with those from the experimental study and effective length approach given by RMI. The finite element model used for the single 2D plane frame was further extended to 3D frames with semi rigid connections, for which the buckling analysis results

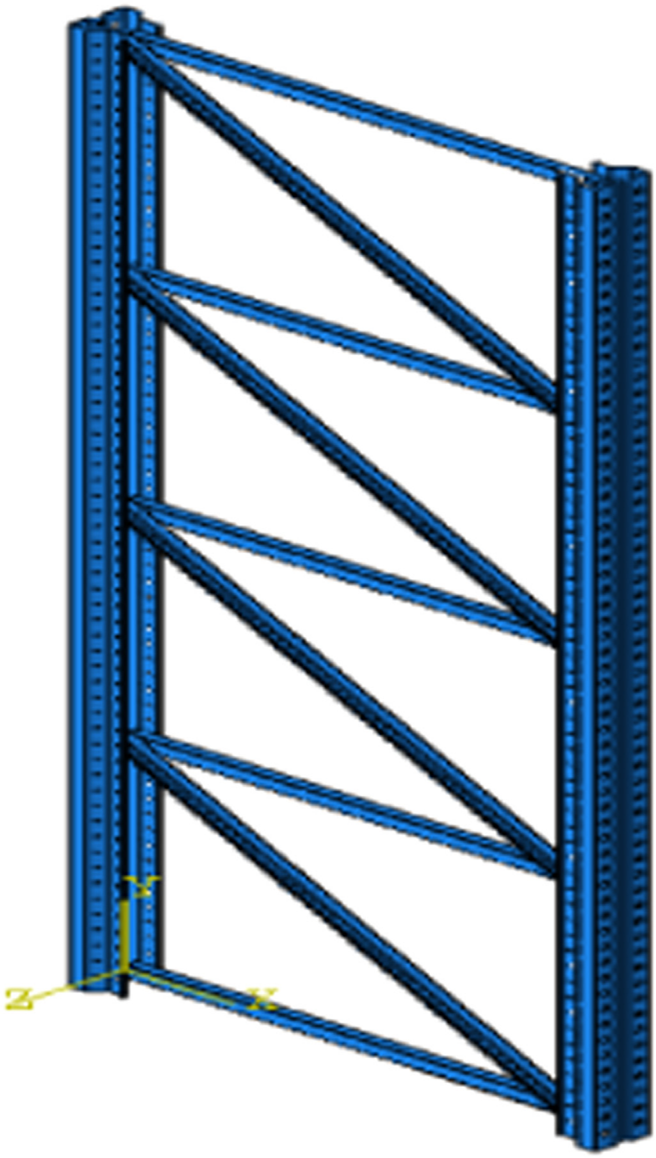


Fig. 7. Bracing Type B2 (horizontal with inclined bracing).

were obtained. However, the study by Sangle et al. [19] does not consider material and geometric nonlinearity in their numerical investigation. Schafer et al. [20] provided an overview of computational modelling, both elastic buckling and nonlinear collapse analysis, for cold-formed steel members and use of the semi-analytical finite strip method and collapse modelling using shell finite elements. Narayanan and Mahendran [21] studied the buckling and ultimate strength behaviour of a series of innovative cold-formed steel members subjected to axial compression. Both laboratory experiments and numerical analyses were used to study the structural behaviour dominated by distortional buckling. Gotluru et al. [22] studied the behaviour of cold-formed steel beams subjected to torsion and bending. The attention was focused on beams subject to torque, because of the effect of transverse loads was not applied at the shear centre. Deshpande [23] studied on buckling and post buckling of structural components such as Shallow Arch, Shallow Truss, Diagonal Truss, Cylindrical Panel and Conical Frusta using ANSYS APDL and ANSYS Workbench. Novoselac et al. [24] presented a linear and nonlinear buckling numerical analysis of a bar with the influence of imperfections. After linear buckling analysis of the bar, they performed a nonlinear buckling analysis by the Riks method. They show that the post buckling behaviour becomes unstable even for a very small value of eccentric load in nonlinear analysis with elasto-plastic behaviour of the material. Casafont et al. [25] worked on the behaviour of steel storage rack columns subjected to compression. Members of different lengths are tested, but focused on the behaviour of specimens having lengths that make them subject to distortional buckling. Rasmussen and Gilbert [26] summarize the main analysis and design provisions of the draft Australian Standard for steel storage racks. The draft standard allows the design to be based on analysis types ranging from linear analysis to advanced geometric and material nonlinear analysis with imperfections (GMNIA). This paper uses standard nomenclatures like LA: Linear (elastic) analysis, LBA: Linear buckling analysis, GNA: Geometric nonlinear analysis, GMNIA: Geometric and material nonlinear analysis with imperfections. Yu [27] provided numerical methods for calculating torsional properties of thin-walled sections. The numerical analysis is performed in FEA software ABAQUS 6.10.

This paper deals with the LA (Linear analysis) and nonlinear buckling analysis of three dimensional 2-D frames of a cold-formed steel storage rack structures, with rigid connections. Initially, linear and nonlinear buckling finite element analysis is carried out on frames made up from 12 types of open column sections. Further, the study is extended on frames made up from 12 types of open column sections strengthened by spacer bars and channel stiffeners to avoid the local buckling of the frames. The results of these studies are presented in this paper.

2. Column sections used in the study

The column (upright) sections in storage racks are perforated for the purpose of easy assembly of the beam end connector. Perforations are generally assumed to decrease the elastic local

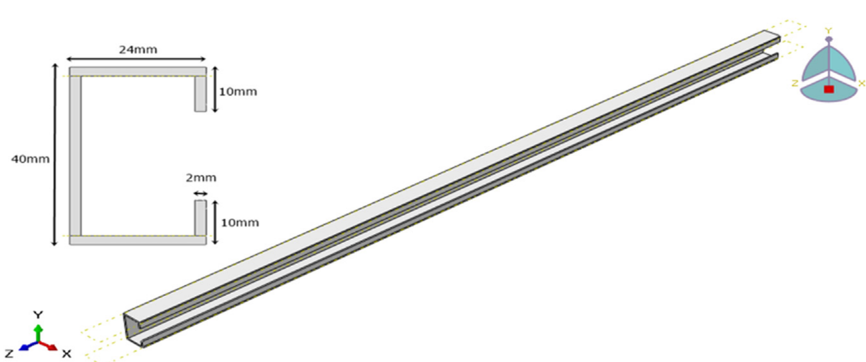


Fig. 8. Details of channel section used as bracing member in modelling of frame with HW column section.

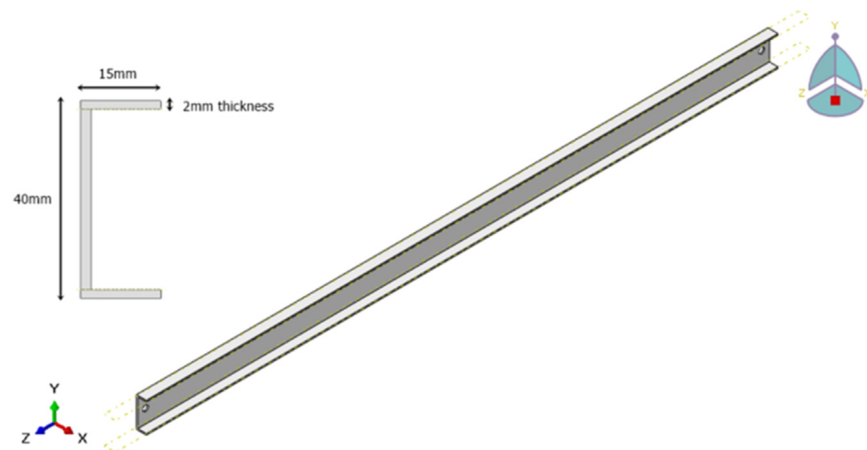


Fig. 9. Details of channel section used as bracing member in modelling of frame with HW column section.

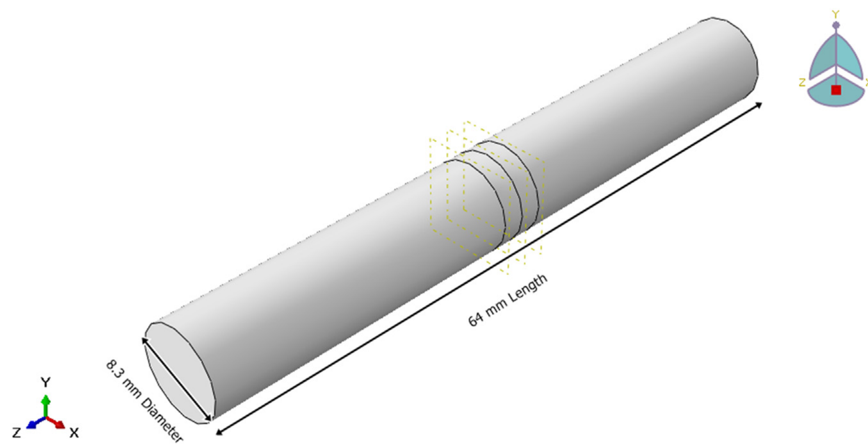


Fig. 10. Details of solid spacer bar used in modelling of MW/HW frame.

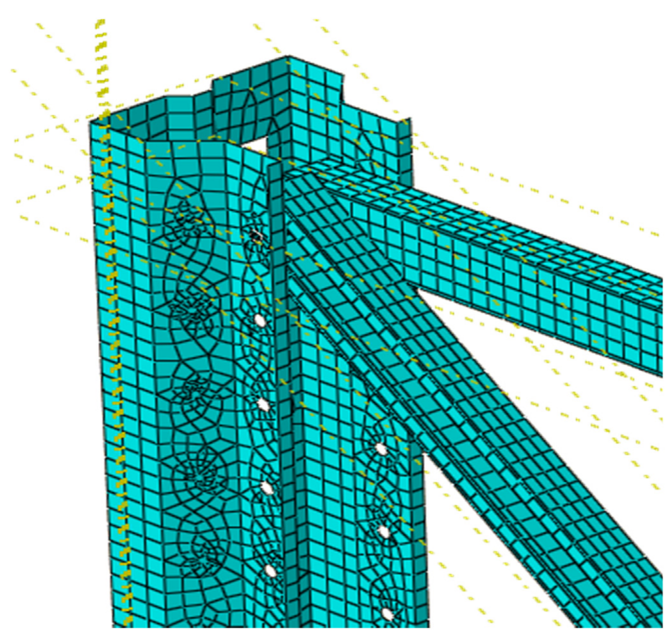


Fig. 11. Typical meshing of column section and details of joint of frame without spacer bar.

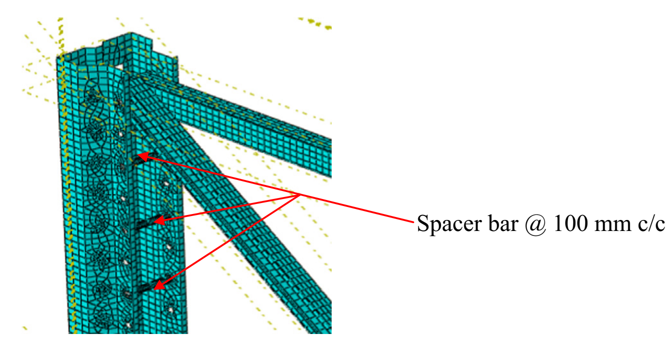


Fig. 12. Typical meshing of column section and details of joint of frame with spacer bar @100 mm c/c.

buckling load of a flat plate loaded in uniform compression; however, hole often causes a change in the wavelength of the buckling mode which actually increases the buckling load away from the hole [28]. The finite element parametric studies demonstrate that holes may create unique buckling modes, and can either decrease or increase a plate's critical elastic buckling stress depending on the hole geometry and spacing [29]. The significance of this increase in strength depends on the geometry and material properties of the member and the boundary conditions. The current specifications allow the use of unperforated section properties to predict the elastic buckling strength of perforated members, by assuming that the presence of such perforations does not have a significant effect on the reduction of the overall elastic buckling strength.

The column (upright) sections used in the study are MW (Medium Weight) column section having three thicknesses 1.6 mm, 1.8 mm, and 2.0 mm each and HW (Heavy Weight) column section

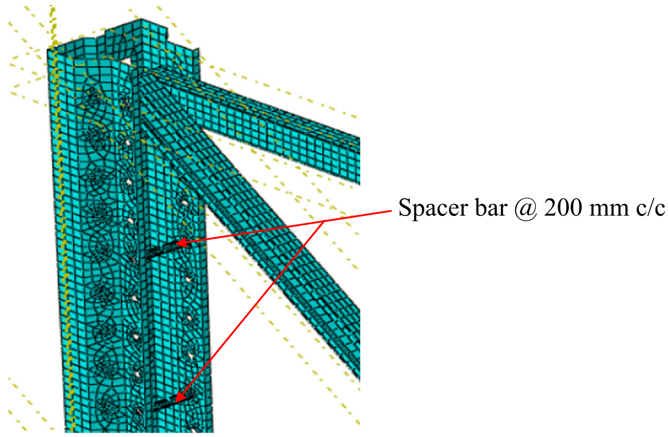


Fig. 13. Typical meshing of column section and details of joint of frame with spacer bar @200 mm c/c.

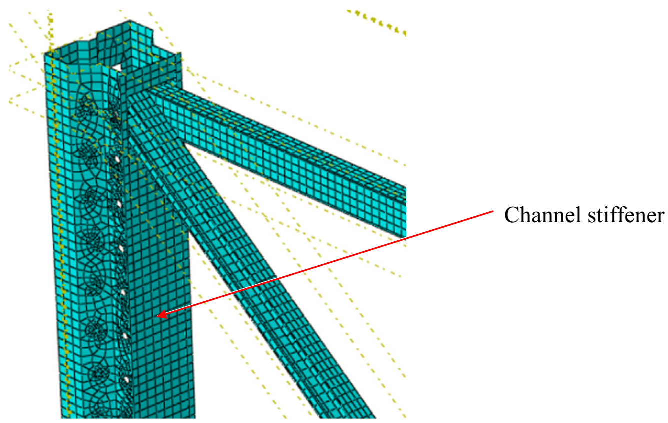


Fig. 14. Typical meshing of column section and details of joint of frame with channel stiffener.

having three thicknesses 2.0 mm, 2.25 mm and 2.5 mm each. Their cross sectional details are provided in Fig. 1 to Fig. 3. Purpose of choosing three different thicknesses is to know the change in behaviour when the sections are made locally stable by having greater thickness. In the present study spacer bars are also provided to avoid the local buckling of uprights. The elastic perfectly plastic (EPP) material behaviour is assumed in the analysis. The material properties of the same sections are given in Table 1.

3. Details of frame

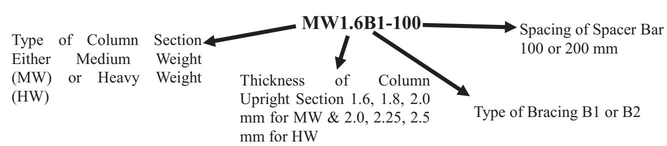
Frame with 1.0 m span has been modeled and analyzed for following cases:

1. Two types of column section HW (Heavy Weight) and MW (Medium Weight)
2. Variation in the thickness (1.6 mm, 1.8 mm, 2.0 mm for MW and 2.00 mm, 2.25 mm, 2.5 mm for HW)
3. Two type of bracing systems B1 and B2 type (i.e. only diagonal bracing and Horizontal with inclined diagonal bracing.)
4. Different spacer bars distances (100 mm and 200 mm)
5. Variation in the frame height (3.1 m, 4.6 m and 6.2 m)

The study of frame divided in to basically 3 types:

1. Basic HW and MW Frames without Spacer bars
2. Basic HW and MW Frames with Spacer bars
3. Torsionally strengthened HW and MW Frame with channel stiffener.

The nomenclature use for frame study is as follows:



4. Finite element modelling and validation

ABAQUS [30], a general purpose FE solver is used for numerical analysis. For all FE models presented in this study, S4R shell element and C3D8R brick elements are used to model columns and bracings respectively. The purpose of using the shell (S4R) and brick (C3D8R) element to model components of a storage rack system is to trace local buckling of elements (flange, web, lip, etc.) of the cross section. Details of these elements are provided in Table 2. Three dimensional Finite Element planer model is validated with experimental results of Sangle et al. [19]. Convergence study is carried on a frame HW2.0B1 of height 3.1 m, to find the proper mesh size of the different parts of the frame such as

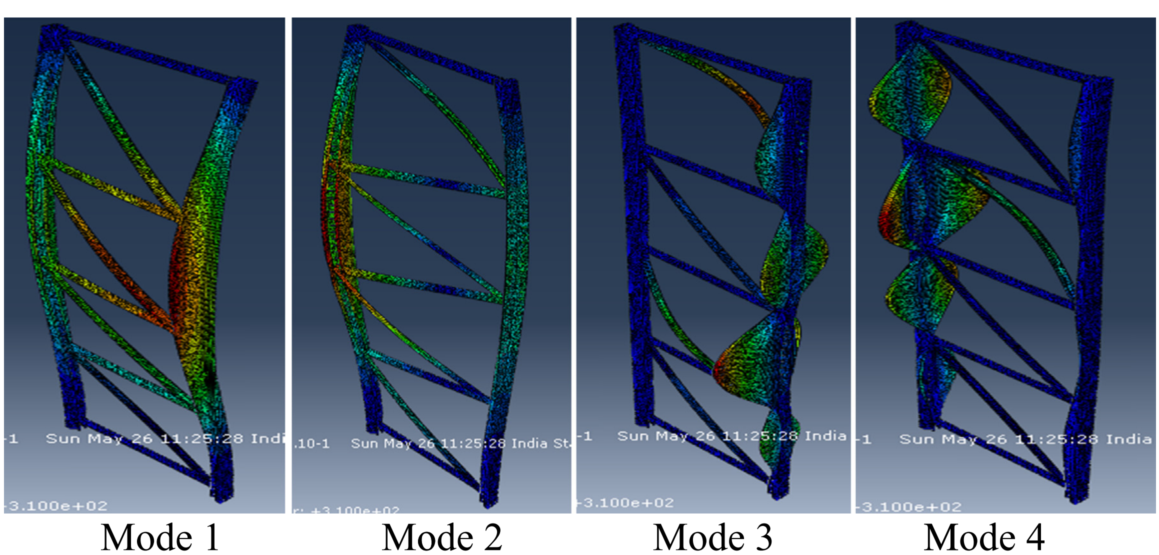


Fig. 15. Eigen buckling analysis modes for Frame MWB2-1.6 mm thickness.

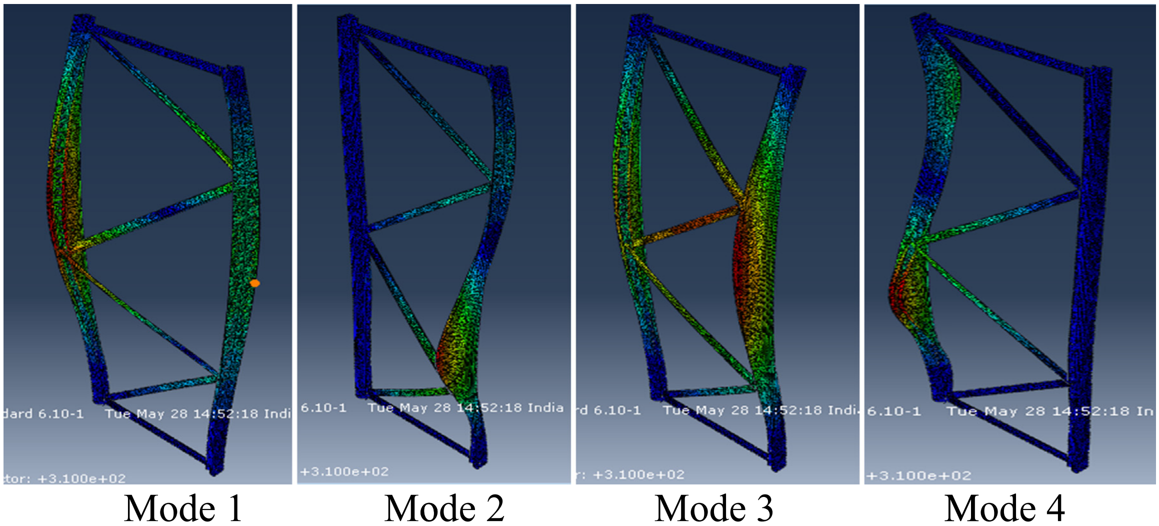


Fig. 16. Eigen buckling analysis modes for Frame MWB1-1.6 mm thickness.

Table 5
FEA results for frames in Section 5.1.

Column frame type	P_e (FEA) for 4.6 m height of frame (kN)		P_e (FEA) for 6.2 m height of frame (kN)	
	Linear	Nonlinear	Linear	Nonlinear
MW-1.6-B1	73.20	65	53.89	50
MW-1.6-B2	88.66	80	61.94	55
MW-1.8-B1	82.39	75	60.31	55
MW-1.8-B2	99.56	95	69.45	65
MW-2.0-B1	91.71	85	66.73	60
MW-2.0-B2	110.38	105	76.92	70
HW-2.0-B1	154.08	145	123.31	115
HW-2.0-B2	194.04	190	154.61	150
HW-2.25-B1	172.16	165	136.93	130
HW-2.25-B2	236.21	230	171.96	165
HW-2.5-B1	190.65	185	150.70	140
HW-2.5-B2	215.21	210	189.13	180

column upright section, bracing and spacer bar, etc. Convergence study presented here is also aimed to validate the FE modelling and analysis techniques in ABAQUS [30]. The results of convergence study are shown in Table 3. From this table it is found that, linear buckling load for mesh size (10 mm × 10 mm) is converging with the experimental value, hence this mesh size is adopted for all analysis presented in this work. Table 4 shows analytical results of FE Models are in good agreement with experimental results [19].

Initially, 120 frames of pallet storage rack system in plane having three different heights, i.e. 3.1 m, 4.6 m and 6.2 m are considered for stability analysis. Details of the finite element models are presented in Fig. 4 to Fig. 10. The frames of rack structure are subjected to compressive load; hence in the model the loads are applied on top of two upright column sections as shown in Fig. 5. The typical meshing of column section and details of joint with and without spacer bars are shown in Fig. 11, Fig. 12, Figs. 13 and 14 with channel stiffener. Details of the various elements used in finite element model are given in Table 3.

The following assumptions are made in FE analysis:

i) The connection between the braces and the columns were considered to be fixed.
ii) At the loading end of the upright all three rotations and displacement allowed and at the bottom base is assumed fixed.

Table 6
FEA results for frames in Section 5.2.

FEA results for frames in Section 5.2 with spacer bars			
Column frame	Spacing in mm	Pe(FEA) kN Linear	Pe(FEA) kN Nonlinear
MW-1.6-B1	100.00	161.87	150
	200.00	140.18	130
MW-1.6-B2	100.00	171.75	160
	200.00	151.28	140
MW-1.8-B1	100.00	189.14	180
	200.00	162.55	150
MW-1.8-B2	100.00	197.10	185
	200.00	173.69	160
MW-2.0-B1	100.00	216.96	200
	200.00	185.52	175
MW-2.0-B2	100.00	232.36	220
	200.00	196.53	185
HW-2.0-B1	100.00	299.12	285
	200.00	269.02	255
HW-2.0-B2	100.00	329.44	315
	200.00	298.97	285
HW-2.25-B1	100.00	347.19	335
	200.00	306.72	295
HW-2.25-B2	100.00	375.65	365
	200.00	338.75	320
HW-2.5-B1	100.00	387.91	375
	200.00	344.91	330
HW-2.5-B2	100.00	422.12	410
	200.00	378.94	365

5. Analysis and results of three dimensional 2-D planer frames

An overall understanding of normal modal analysis as well as knowledge of the natural frequencies and mode shapes of structure is important for all types of analysis. Eigen value analysis is the basis for many types of analyses. Eigen value analysis of storage rack systems is carried out by using ABAQUS, as a general purpose FE platform which is based on solution of following stability Eq. (1).

$$\left([K]_{n\times n} - \lambda_n [K_G]_{n\times n} \right) [x]_{n\times 1} = 0 \qquad (1)$$

where $[K]_{n\times n}$=stiffness matrix of the structure λ_n=buckling Eigen value corresponding to nth mode $[K_G]_{n\times n}$=geometric stiffness matrix of the structure $[x]_{n\times 1}$=mode shape vector of nth mode.

Table 7
FEA results for frames in Section 5.3.

Section 5.3 using Channel Stiffener				
Column frame	P_e in kN (Experimental)	P_e (FEA) in kN	P_e (FEA) in kN	% error
		Linear	Nonlinear	P_e Linear
MWC-1.6-B1	155.26	164.95	160	−6.24
MWC-1.6-B2	165.28	173.98	165	−5.26
MWC-1.8-B1	246.48	263.46	255	−6.89
MWC-1.8-B2	266.48	268.81	260	0.87
MWC-2.0-B1	320.65	298.95	290	6.76
MWC-2.0-B2	335.62	294.1	285	12.37
HWC-2.0-B1	357.52	412.12	400	−15.27
HWC-2.25-B1	422.78	455.43	445	−7.72
HWC-2.25-B2	435.52	465.89	455	−6.97
HWC-2.5-B1	475.69	491.76	485	−3.37
HWC-2.5-B2	483.56	503.56	490	−4.13

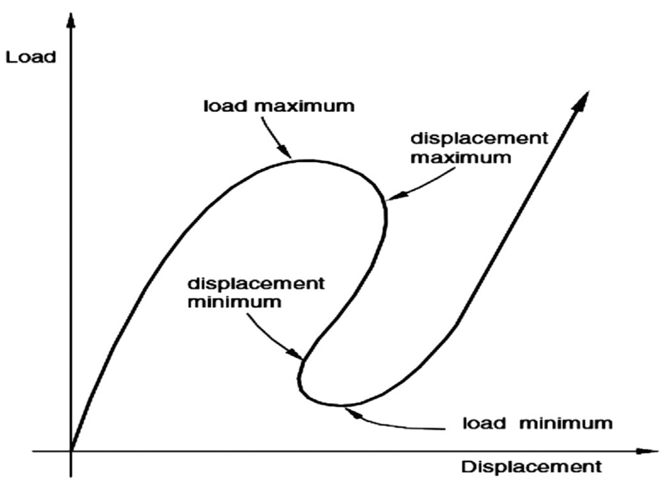


Fig. 17. Typical unstable static response [24].

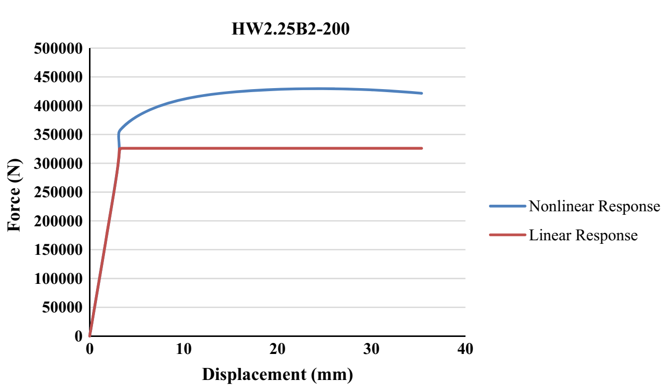


Fig. 18. Linear and nonlinear buckling response of HW2.25B2-200 frame.

ABAQUS uses two different approaches to solve the above equation: SUBSPACE (default) and LANCZOS. The Subspace Iteration Method (a classical method and by default Eigen solver in ABAQUS) introduced by Bathe [31] is used for the analysis presented here. The linear buckling (Eigen value) analysis is performed for determination of critical buckling load and first three buckling modes using software tool ABAQUS. Most of the frames have the same type of bucking shapes. For different frames following buckling mode shapes are shown.

Mode 1: Sway in down-aisle direction.

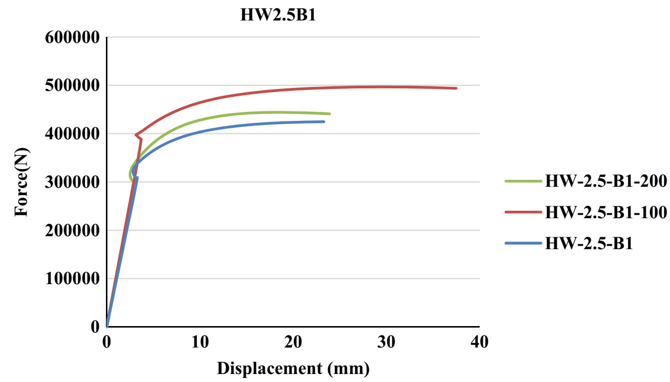


Fig. 19. Nonlinear buckling response of frame HW2.5B1 with and without spacer bars.

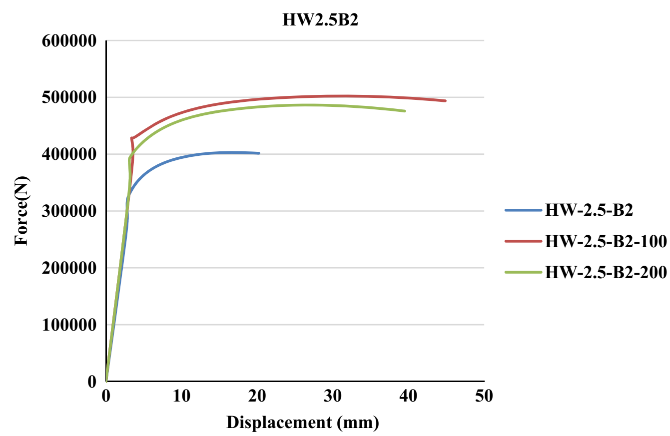


Fig. 20. Nonlinear buckling response of frame HW2.5B2 with and without spacer bars.

Mode 2: Torsion.
Mode 3: Local buckling or sway in 2nd mode.
Mode 4: Local buckling or sway in 3rd mode.

5.1. Study 1

This study includes one load case, i.e. compressive load as shown in Fig. 5 and two types of bracing pattern as shown in Figs. 6 and 7. Finite element analysis (both LA and nonlinear buckling) is conducted for 12 types of column sections with 3.1 m height. The critical buckling shapes are found from Eigen value buckling analysis in ABAQUS. The same failure shapes are found after a nonlinear buckling analysis. Few buckling shapes are summarized in Figs. 15 and 16. Also the linear and the nonlinear buckling response of the frame are summarized in Table 4.

The finite element result shows a good agreement with experimental results [18]. Hence the study is further extended to two more heights of frame, i.e. 4.6 m and 6.2 m. The results of the LA and nonlinear buckling analysis of these frames are shown in Table 5.

5.2. Study 2

Finite element analysis is conducted for same frames as in Section 5.1 with spacer bars. The spacing of spacer bars is kept at 100 mm and 200 mm and the results are summarized in Table 6.

5.3. Study 3

In Section 5.3, both MW and HW column sections are strengthened

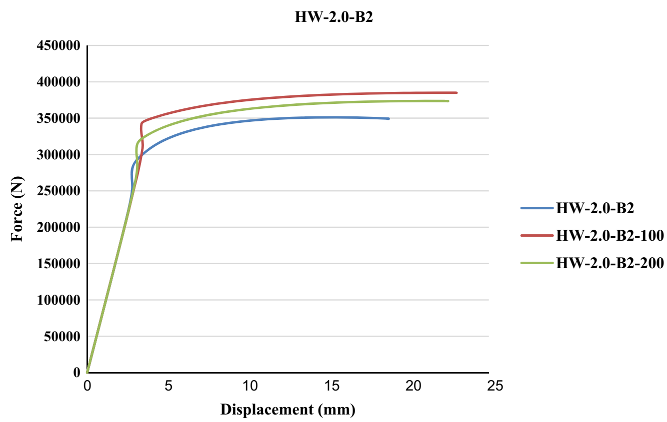


Fig. 21. Nonlinear buckling response of frame HW2.0B2 with and without spacer bars.

by adding external stiffeners i.e. channel stiffener as shown in Fig. 14. The finite element results are summarized in Table 7.

6. Nonlinear buckling analysis

Nonlinear buckling analysis with material nonlinearity and the effect of plastification is used to investigate post buckling behaviour. Since the nonlinear buckling behaviour may become unstable when the elasto-plastic deformations take place, it is very important to investigate the influence of imperfections on the frame loading capacity. Geometrically nonlinear static problems sometimes involve buckling or collapse behaviour, where the load-displacement response shows negative stiffness and the structure must release strain energy to remain in equilibrium. The Riks method uses the load magnitude as an additional unknown; it solves simultaneous for loads and displacements [24]. For unstable problems, the load displacement response can exhibit the type of behaviour shown in Fig. 17. That is, during periods of response, the load and/or the displacement may decrease as the solution evolves.

Therefore, for nonlinear analysis 'Static Riks' method is suitable for predicting buckling, post-buckling, collapse of highly nonlinear of structures where linear-based Eigen value analysis will become inadequate. In these analyses, the transfer from stable to unstable state is investigated.

In the present study to investigate nonlinear buckling behaviour of storage rack structures, Finite Element models are analyzed in Static Riks step with geometric nonlinearity on (Nlgeom: ON). This analysis is controlled by force and terminated when LPF (Load Proportionality Factor) is negative. From the nonlinear buckling analysis, various failure modes are observed. In Fig. 18 to Fig. 21 shows the nonlinear buckling analysis response of various frames.

Fig. 18 shows the linear and the nonlinear buckling response of frame HW2.25B2 with spacer bars at 200 mm spacing. From nonlinear buckling results show that the critical nonlinear buckling loads are less than the linear. The estimation of the critical buckling load is based upon the results as shown in Fig. 18. For nonlinear buckling FEM model which show that bifurcation point is approximately at 320 kN. The nonlinear buckling behaviour of the frame with ideal load shows stiffness decrease after bifurcation point. The results show that when the frame enters in the elasto-plastic condition, there is a significant decrease of critical buckling load compared to a linear model. The force verses displacement graphs are plotted for various frames with and without spacer bar. Each graph contains the nonlinear buckling response of the frame without spacer bar (e.g. HW-2.5-B2), spacer bars with 100 mm spacing (e.g. HW-2.5-B2-100) and spacer bars with 200 mm spacing (e.g. HW-2.5-B2-200). The nonlinear buckling responses of some of the frames are presented in Fig. 19 to Fig. 21.

7. Conclusions

Numerical studies have been performed in the present work to investigate linear and nonlinear buckling failure modes of frames of rack structures. Following significant conclusions of the studies are summarized as below:

- Linear Eigen value analysis can be used for calculation of critical buckling load of the structure.
- For evaluation of the nonlinear buckling response, the Riks method in ABAQUS can be used.
- From the numerical study, it is found that spacer bars are effective in enhancing the strength of these cold formed pallet rack frames.
- Buckling loads of frames from finite element analysis show good agreement with experimental results.
- Marginal difference is observed between frames with horizontal and diagonal bracing system and frames with only diagonal bracing system.
- The channel section used as a stiffener increases the buckling strength of the frame.
- Reduction in buckling load is observed after increasing the height of the frame due to increased slenderness.
- Hat sections configuration by shape and size wise need to be revised so as to avoid local and torsional buckling modes.
- Yielding of material transforms the stable post buckling behaviour into unstable. An increase in the displacement causes the decrease of the corresponding load carrying capacity after yielding.

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