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Comparative Design of an Industrial Building: Cold Formed Steel versus Hot Rolled Steel

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Abstract

Industrial buildings made of Cold Formed Steel (CFS) members have gained an extent of construction all over the world. It has become an alternative process of construction to the Hot Rolled Steel (HRS) elements, allowing it to respond to the requirements in a short time. The aim of this work is to present a technical and economical comparative study of an industrial building case made of CFS versus HRS elements and designed according to the European and American codes. Relying on numerical modeling, the design according to Eurocode 3 demonstrates that the industrial building made of CFS is more economical compared to the HRS members building by 43% in terms of weight and 28% in terms of cost. In addition, based on the Average Capacity Design (ACD) ratio of the CFS elements, it turns out that the design according to Eurocode 3 part 1-3 is less conservative than the same building designed according to the American Iron and Steel Institute (AISI) code. Moreover, the American code privileges the safety aspect of the designed CFS building with the ASD method over the LRFD and LSD methods. Otherwise, the CFS building designed according to Eurocode 3 part 1-3 with truss columns and beams reveals a weight saving of 25% and 14%, respectively, compared to the building with a full web of beams and columns, and the building with a full web of columns and steel trusses.

Keywords

AISI, Cold-Formed Steel (CFS), Eurocode 3, Hot-Rolled Steel (HRS), Industrial building, Numerical modeling

1. Introduction

Nowadays, Cold Formed Steel (CFS) structures have become a promising alternative to the conventional Hot Rolled Steel (HRS) ones (Lim et al. 2016; Martınez 2007; Harini et al. 2020; Harshavardhan et al. 2021; Jayaraman et al. 2018). Generally, these types of structures are widely used in housing, scholar buildings, and hospital buildings with an interesting saving in cost and time in construction (Ortiz 2020; Balh 2010, Fatimah et al. 2023). Besides, industrial buildings made of cold formed steel elements, such as warehouses, have attracted more interest in recent years (Early et al. 2018; Rehman and Sakalle 2019; Vujanac et al. 2017). Due to their lightweight, an important reduction in the load's transmission is noticed from the superstructure to the foundation, which presents a significant advantage compared to the HRS (Kankuntla et al. 2018). Consequently, a positive effect on the foundations' design and dimensions is resulting (Goswami and Shende 2018). Moreover, the process of manufacturing CFS elements from steel coils can be carried out on site using easily transportable bending or profiling machines (Hancock 2016). This allows these kinds of buildings to become more economical by creating a manufacturing workplace on site (Schafer 2011; Stsepaniuk et al. 2021).

However, due to the thin thickness of CFS elements that are commonly called light gauge steel, buckling and local buckling instabilities are often prevalent in these elements under axial compressive load (Rouaz et al. 2018,2019; Hancock 2003; Bešević et al. 2017; Rouaz et al. 2020; Hancock 2016). Thereby, numerous research projects have been carried out to address these issues. D. Wang et al. (2020) have performed an optimization study of the resistance of CFS stiffened elements under compressive stresses. Zhou et al. (2022) have developed a new methodology to evaluate the buckling resistance of the CFS columns under compression and bending loads. Furthermore, the study conducted by Deepak et al. (2021) revealed that the use of double I-beams or box-beams that behave as a unique element increased the local buckling resistance of the CFS members. Moreover, prediction and verification methods for these kinds of instabilities have been implemented by Mahar et al. (2022) for unstiffened steel elements. Nevertheless, checking the resistance of the cross-sections of each CFS element of the building must be carried out, considering the effective width of the element in the structure's design.

Indeed, mathematical and empirical formulations developed through several research works and provided by various codes and standards: European code (Eurocode 3), American code (AISI S100-16), Australian code (AS/NZS 4673) and the British standard (BS 5950-5:1998) are widely implemented in engineering computing software. First, this allows to compute the effective width of the usual CFS elements (Yu et al. 2019) based on the developed internal effort that causes the compression of the element's width. Then, it also allows to undertake the verification of the resistance and rigidity of the CFS elements to design the structure's members of the industrial buildings. In this perspective, this work aims to promote the construction for industrial buildings with steel frames based on cold formed steel elements, in addition to industrial buildings made of hot rolled steel elements. First, a brief overview is presented of the international normative aspects and codes, about their application to the design of these industrial buildings. Then, a numerical study based on Finite Element Modeling (FEM) is performed to design a single-store warehouse as an industrial building case using SAP2000 v24 software. CFS and HRS industrial buildings are both modeled and designed using Eurocode 3 P1-1, 3 (EN 1993-1-1, 3) for weight comparative purposes. A second comparison is performed between two CFS industrial buildings designed using the Eurocode 3 and the AISI code, in terms of weight, Average Capacity Design (ACD) and Individual Capacity Design (ICD) criterion, under wind loads.

2. Research study methodology and code design

The design study of the industrial steel building according to the different codes and methods, which consists of checking the resistance of the elements, their rigidities, as well as its overall stability is presented in Fig. 1. The Eurocode 3 P1-1 (EN 1993-1-1) is applied to design the industrial building made of HRS elements, and the Eurocode 3 P1-2 (EN 1993-1-3) concerns the same building but made of CFS elements. However, the American Iron and Steel Institute (AISI) is used to design the building's cold formed steel elements with its three methods, namely:

- The Allowable Stress Design (ASD) method, also referred to as "service load design." It is a design methodology based on the verification that the maximum stress developed in any structural element must always be smaller than a certain allowable stress in service conditions, which is defined according to its nominal strength over the safety factor (Ω). On the other hand, the structural element's behavior is limited to the elastic state (the serviceability limit state).
- The Load and Resistance Factor Design (LRFD) method. This method considers the uncertainties in the determination of the

loads and the resistances by applying two safety factors: one to decrease the resistance and the other to increase the different loads. Therefore, the resistance of the structural element can achieve its ultimate limit state.

- The Limit States Design (LSD) method, like the LRFD, is based on realistic loading conditions and material properties of elements and considers both limit states with different values of safety factors.

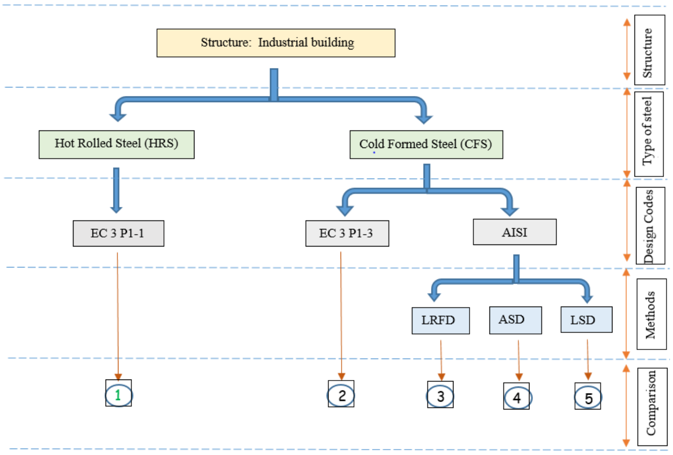


Fig. 1 Flow chart of the study

3. Numerical modeling considerations and design criterion

As regards the industrial building, a single-store warehouse located in Cherchell (Tipaza, Algeria) is selected as a case study and modeled with the finite element method. The end wall's (width) and side wall's (length) dimensions are 20 m and 48 m respectively. The height of this warehouse at the eave strut is 6 m, and the ridge height (top of the rafter) is 7.85 m, as illustrated in Fig. 2a. This warehouse is composed of duo pitch roofs, of which has an angle equal to 10.5°. The thicknesses of the sandwich panels covering the building as a wall and roof are 35 mm (LL35) and 75mm (TL75), respectively. The spacings of the purlins and girts of the building are, respectively, 1.37 m and 1.6 m. The warehouse contains several openings: a 6 m x 5 m opening in the middle of each side wall and a 5 m x 5 m opening at each end wall. However, wall and roof bracing systems are planned for this industrial building as a lateral force resisting system under wind load. Both wind and snow loads are determined according to the Regulatory Technical Document of Snow and Wind (RNV 2013 DTR-C-2-47) while the temperature load is assessed using the Regulatory Technical Document CCM 97. Furthermore, all elements' weights in this warehouse are considered as Dead Loads (DL) in the finite element modeling. The Wind Load (WL) and Temperature Load (TL) are added to the dead load in different combinations, with coefficients of ponderation, to get the most ultimate case for the design.

However, the mechanical properties, introduced in the finite element modelling, of cold formed steel elements are assumed to be like those of hot rolled steel. Hence, the yield stress Fy is 344 MPa, the tensile stress Fu is 448 MPa, and the elastic modulus Es is assumed to be 2.1 105 MPa. The density is 7850 Kg/m3.

To simulate the real conditions of the constructed building, a few assumptions in the numerical modeling such as the boundary conditions of the elements, type of the element assembly, buckling and/or lateral buckling lengths, are considered in the numerical modeling. The parameters of each modelled element for HRS and CFS building are defined in Table 1, before performing the verification of the resistance and rigidity of the elements by the check design option of the SAP2000 V24 software.

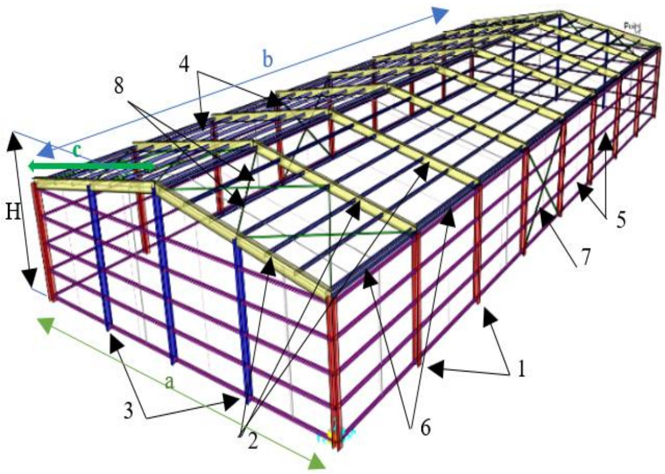
Where the signification of the used letters is as follows:

- N: no solicitation;
- M2, M3 and P are bending moments around, respectively, the Z-axis (Axis 2), and the Y-axis (Axis 3);
- P: is the axial force;
- Ly: is the buckling length around the Y-axis (Axis 3);
- Lz: is the buckling length around the Z-axis (Axis 2);
- n: is the purlin spacing.

All the analyses of the design and dimensions of CFS and HRS of the industrial building elements throughout this numerical study are based on the Capacity Design (CD) criterion, which represents the sum of the ratio of the developed stresses over the resistance stress of the most stressed steel element.

To provide stability to the industrial building in the end wall direction, the assembly of each column with the rafter is assumed to continue, so that they form a constrained portal in the width direction (end wall) of the building. A resistance moment is expected to develop at the base of the column around axis 2 and axis 3, as depicted in Fig. 2b. However, the posts (secondary column) elements are installed in the end wall direction to receive only the loads of the cladding girt owing to the considerable span. The assembly of these posts is considered pinned to the steel rafter and to the base of the building. Hence, a Release (R) of the resistance moment M2, M3, and the vertical load (P) that may arise from the rafter is configured in the numerical model. While the remaining horizontal elements such as the purlin, eave strut, and grift, as well as the wall and the roof bracings stability, are considered released in the resistance moment.

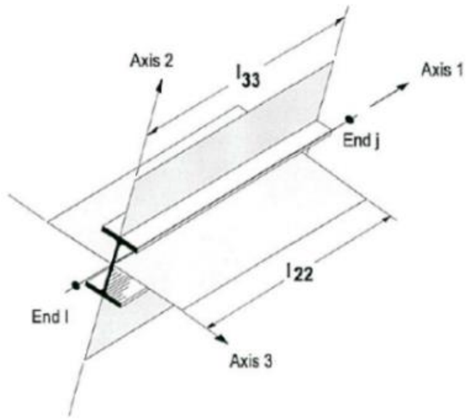
In addition, lateral supports for the upper flange of the girts and purlins are intended for the realization of this building. Therefore, the verification of the structural rigidity of these elements against the lateral buckling is not considered in the numerical modeling. Otherwise, the buckling length of the steel column is divided by the number of girts, and that of the purlin and girt by half-span (2).



1. Column
2. Rafter
3. Post
4. Purlin
5. Girt
6. Eave strut
7. Wall bracing
8. Roof bracing

a. End wall
b. Sid wall
c. Slop roof

(a) Steel warehouse elements



(b) Local axes of modelling elements (SAP 2000)

Fig. 2 Finite elements modelling

Table 1. Model configuration

Elements	Release			Buckling length		Lateral buckling length	Lateral support	
	M_2	M_3	P	L_y	L_z		Sup	Inf
Column				L	L/n	L		
Rafter				L	L	L/2		
Post	N	N	N	L	L	L		
Purlin	N	N		L	L/2	L	N	
Girt	N	N		L	L/2	L	N	
Eave strut	N	N		L	L	L		
Wall Bracing	N	N		L	L	L		
Roof Bracing	N	N		L	L	L		

4. Results and discussion

In accordance with Eurocode 3 P1-1 methodology, the HRS structure is designed and analyzed by comparing multiple model configurations to determine the optimal HRS design, which is then compared to the CFS structure. A scheme of the methodology that leads to the selected HRS model is presented in Fig. 3. Several linear analyses are carried out from the model M.0.0 to M.0.6, except for the model M.0, which is performed with a nonlinear analysis.

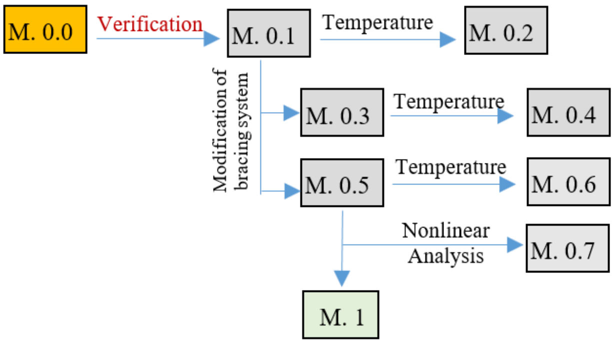
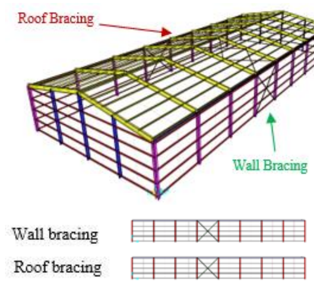
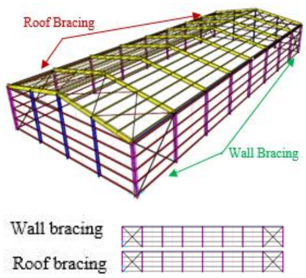


Fig. 3 Strategy for the selected HRS model

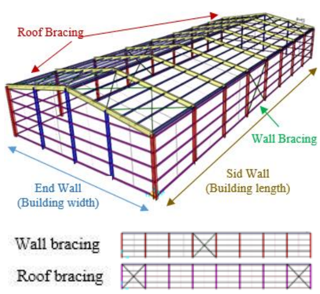
The model M.0.0 represents the basic model developed from a flat-rate method of dimensioning, containing only one X-shape bracing on the two lateral side walls and two T-shape bracings on each side of the roof, as illustrated in Fig. 4a. The verification of both resistance and stability of this first model has led to the development of a second model M.0.1. In this model (M.0.1), the uniform variation of the temperature's loads is introduced to study its effects on the steel elements design, resulting in the model M.0.2. The positions of the wall bracing and the roof bracing systems in model M.0.1 are modified and studied in the model M.0.3, as shown in Fig. 4b. However, the temperature effects on the HRS elements of the model M.0.3 are studied in the model M.0.4. Moreover, the positions of the wall bracing systems in the model M.01 have been shifted once again to get M.0.5, as shown in Fig. 4c, to study their effects on the design of the model M.0.5 compared to the model M.0.4. In the model M.0.6, the effects of the variation in the temperature loads are investigated based on the model M.0.5, while the model M.0.7 presents a nonlinear analysis of the model M.0.5. Finally, model M.1 is the selected model that has been used in the design analysis of the HRS structure.



(a) Models: M.0.0, M.0.1, M.0.2



(b) Models: M.0.3, M.0.4



(c) Models: M.0.5, M.0.6, M.0.7, M. 1

Fig. 4 Localization of wall and roof bracing systems

The cross-sections of all HRS elements for each model are summarized in Table 2. The design of the HRS elements of the initial model M.0.0 that has been realized based on a flat-rate dimensioning method shows an average Capacity Design ratio equal to 1.22. To reduce this ratio, the individual Capacity Design ratio of each element must be reduced as well. Therefore, except for the eave strut and a small changing of the bracing elements of the cross-section, all elements' sections of this building (M.0.0) have been increased to achieve an individual CD ratio less than 1.

Fig. 5 presents the numerical results of the maximum individual CD ratio of each element under for than 200 load combustions. The Average CD ratios of the models M.0.1 and M.0.2 are equal to 0.707 and 0.710, respectively. This comparative result demonstrates that the uniform variation of the temperature load in the warehouse steel elements, with wall bracing and roof bracing systems located at the middles of the side wall and roof (Fig. 4a), has no effect on the dimensions of the HRS elements. While, when the wall bracing and roof bracing systems are located at the end of side wall and roof (Fig. 4b), as the model M.0.3, the average CD ratio decreases by 2. However, the results reveal that the uniform variation of the temperature load introduced in the model M.0.3 has an important impact on the individual CD ratio of the eave strut (model M.0.4) in which it was increased from 0.48 to 1.45. This is because the wall bracing components have limited the beam's number of degrees of freedom, which has led to the development of a compressive stress in this element. In fact, at positioning the wall bracing members at the middle of the side wall in model M.0.5, the individual CD ratio of the eave strut beam decreased to 0.71. In addition, the computed average CD ratio of the model M.0.5 is equal to 0.73 after having been equal to 0.85 in the model M.0.4, which means that this average CD ratio has decreased by 8%.

As regards the models M.0.6 and M.0.7, the average CD ratios seem not to be affected by the addition of the temperature loads and the nonlinear analysis, reactively. Otherwise, the individual CD ratio of the wall bracing members has increased by 6% compared to the individual CD ratio obtained while performing linear analysis, since these wall bearings receive all the axial tensile forces in the nonlinear analyses. Finally, the configuration M.1 of the HRS warehouse elements, corresponding to the Model M.0.5, is maintained for comparison with the same configuration but with cold formed steel elements.

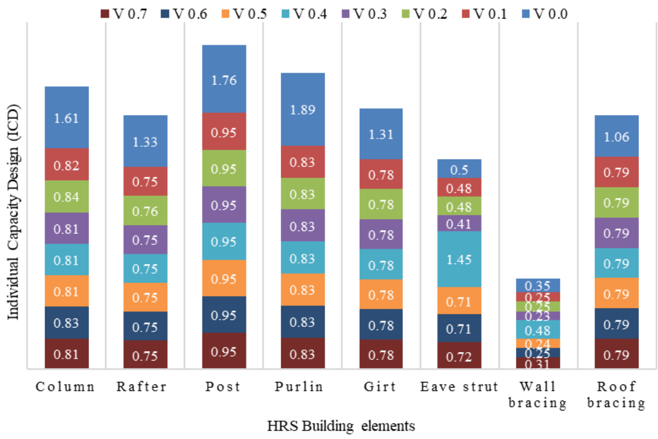


Fig. 5 Individual CD ratio of the HRS elements

4.1 Technical-economical evaluation

Comparative study between the HRS and CFS warehouse design according to Eurocode 3

The developed model M.1-HRS has been designed according to the Eurocode 3 P1-1, whose elements have been replaced by CFS elements to build the model M.1-CFS. Hence, all the resistance, rigidity of elements, and the global stability of M.1-CFS are checked using the SAP2000 steel check-design according to Eurocode 3: P1-3. The cross-sections of these CFS elements are listed in Table 3.

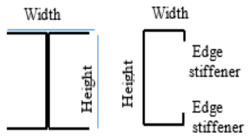
Fig. 6 shows the comparison between M.1-CFS and M.1-HRS in terms of the total weight of each element. It turns out that all hot rolled steel elements have a greater total weight than those made of cold formed steel. This confirms the advantage of thin-thickness CFS elements, which provide a gradual increase in the thickness of the element, unlike the available and imposed thicknesses of standard HRS elements. Moreover, the computed average CD ratio of both HRS and CFS buildings is equal to 0.73 and 0.83, respectively. This means that the use of the CFS elements is more optimal than the HRS components.

In terms of weight, the designed hot rolled steel elements have a total weight of 57 tons, while the total weight of the cold formed steel elements is 32 tons. This leads to conclude that the warehouse made of CFS is more economical, in terms of weight, by 43% compared to the same building made of HRS elements. However, based on these results, comparing the CFS elements to the same HRS elements, it is evident that the CFS elements subjected to large axial compression stresses, such as the columns, do not exhibit a discernible weight savings.

Table 2. Different models elements

Models	Elements							
	Column	Rafter	Post	Purlin	Girt	Eave strut	W-Bracing	R-Bracing
M.0.0	IPE400	IPE400	IPE240	IPE120	UPN120	HEA120	2xL70x7	L50x5
M.0.1 to M. 1	HEA300	IPE500	IPE300	IPE160	UPN160	HEA120	2X60x6	L60x6

Table 3. Dimensions of the M.1-CFS sections according to EC 3: P1-3

CFS sections	Elements	Height [mm]	Width [mm]	Thickness [mm]	Edge stiffener [mm]	Section	
2x 400S170-500	Column	400	2x170	5.0	/		
2x 400S170-500	Rafter	250	2x170	5.0	/		
2x 250S170-350	Post	250	2x170	3.5			
200S76-250 (+16)	Purlin	200	76	2.5	16		
150S76-250 (+16)	Girt	150	76	2.5	16		
150S76-250 (+16)	Eave strut	150	76	2.5	16		
100S51-150	Wall bracing	150	51	2.5	16	Column, rafter and post	Rest of elements
100S51-150	Roof bracing	100	51	1.5	/		

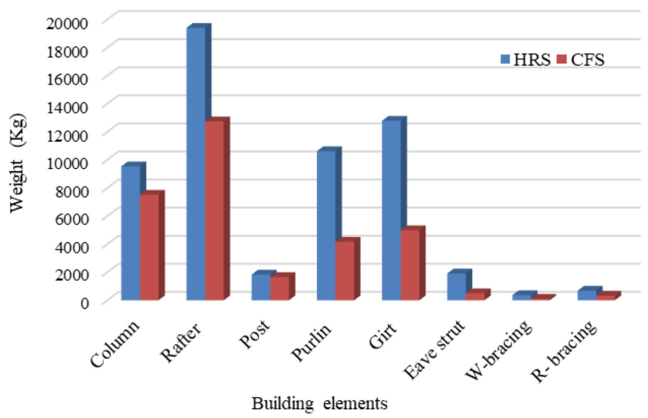


Fig. 6 Elements weight of both HRS and CFS building designed using the EC3

Comparative study between CFS structures design using Eurocode 3 and AISI

The second part of this study compares the design of the M.1-CFS model, having the geometrical characteristics and elements' configuration as presented in Fig. 4-c, according to Eurocode 3 and the AISI (American Iron and Steel Institute) design methodologies. The resistance, rigidity, and stability of all the structural elements, as well as the global displacement at the top of the steel warehouse, are also checked according to the AISI code. Table 4 indicates that the M.1-CFS model designed with Eurocode 3: P1-3 produces a higher average CD ratio than the one calculated for the identical model designed with the three AISI approaches. This leads one to conclude that, in comparison to the American code, the European code is less conservative. In addition, the cross-sections of the designed M.1-CFS building according to the AISI design codes can be reduced, which could provide a lighter weight.

However, the three AISI design approaches present a significant variation in terms of the average CD ratio. A difference of 15% and 26% in the average CD ratio is observed by comparing the LSD method to the LRFD and ASD, respectively. It can be explained through the inversely proportional relationship that each design method is established regarding safety and economic criteria. From a mathematical formulation point of view, for each loading condition, the nominal resistance is divided by a coefficient (Ω) larger than 1 for the LSD methodology, and it is multiplied by a coefficient (ϕ) smaller than 1 for the LRFD and LSD methodologies. This indicates that the ASD method works at the serviceability limit state, while the LRFD and LSD methodologies work at the ultimate limit states. Thereby, the ASD methodology of the AISI code is more conservative than LRFD and LSD, respectively, with a difference of 11% and 26%.

4.2 Optimal design of the CFS structure according to Eurocode 3

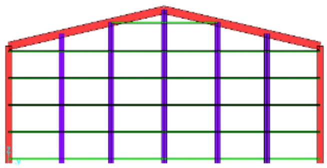
The results presented above reveal that the European design methodology is the most economical approach in terms of the average CD ratio criterion (§ 4.1.2). In this context, the objective of this current part of this study is to find out the optimal design of structure's elements in terms of economy by analyzing three models' configurations designed according to Eurocode 3: P1-3. As the columns and the rafters constitute the main elements of the steel frames that support gravity loads and assure the most significant part of the structural rigidity and stability of the building, two different configurations of the model M.1-CFS are developed. Thereby, the structural elements of the M.1-CFS that are composed of full web cross-sections of the main column and rafter, as illustrated in Fig. 7-a, are substituted by truss elements. Therefore, the model M.2-CFS, presented in Fig. 7-b, is made of full web C cross-section columns and truss rafters. While the columns and rafters of the model M.3-CFS are entirely composed of truss elements, as shown in Fig. 7c.

Table 4. Maximum individual and average CD ratios of the M.1-CFS elements

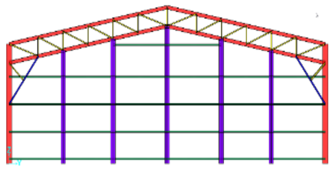
Elements	CFS	EC3 : P1-3	AISI code		
			ASD	LRFD	LSD
Column	2x 400S170-500	0.99	0,51	0.68	0.85
Rafter	2x 400S170-500	1.00	0.50	0.68	0.85
Post	2x 250S170-350	0.90	0.39	0.58	0.80
Girt	150S76-250 (+16)	0.94	0.31	0.43	0.61
Purlin	200S76-250 (+16)	0.87	0.34	0.43	0.53
Eave strut	150S51-250 (+16)	0.90	0.42	0.59	0.82
wall bracing	100S51-150	0.46	0.28	0.30	0.41
Roof bracing	100S51-150	0.59	0.38	0.35	0.35
	Average CD ratio	0.83	0.39	0.50	0.65

Table 5 presents the sections of the CFS elements of the models M.2-CFS and M.3-CFS together with the calculated individual CD ratio for each element. Based on the results obtained through numerical analysis, the average CD ratio of both M.2-CFS and M.3-CFS models is equal to 0.77 and 0.74, respectively. These two ratios are close to the average CD ratio computed for the model M.1-CFS, which is 0.83. This means that the modification of the type of the structural element from a full web to truss elements could lead to a quasi-unchanged average CD ratio of the entire structure while the structure is designed using the same design code (in this case, Eurocode 3: P1-3).

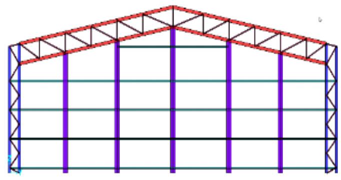
However, in terms of building weight, the total weight of the three modeled structures, M.1-CFS, M.2-CFS, and M.3-CFS is respectively around 32 tons, 29 tons, and 24 tons. This result demonstrates that the configuration with truss columns and rafter's elements (M.3-CFS) is the most economical configuration in terms of weight, with a difference of 14% compared to the building with truss rafters and full web columns, and 25% when the columns and the rafters are made of truss elements. This is because the buckling length of the full web column is more important than the buckling length of the truss, leading to a larger section that is less economical.



(a) M.1-CFS: Full web columns and rafters



(b) M.2-CFS: Full web columns and truss rafters



(c) M.3-CFS: Truss columns and rafters

Fig. 7 Different CFS model configurations

Table 5. CFS elements of two the model configurations and their individual CD ratios

CFS Elements	M2-CFS		M3-CFS	
	Section	ICD	Section	ICD
Column (full web)	2x 350S150-400	0.94	//	//
Column (chord and post)	//	//	2x 150S76-300	0.92
Column (Diagonal members)	//	//	100S51-200 (+16)	0.98
Rafter (chords)	2x 180S76-400	0.94	2x 150S76-300	0.91
Rafter (Post et diagonal)	100S51-200 (+16)	0.91	100S51-200 (+16)	0.58
Post	2x 250S170-350	0.90	2x 200S140-300	0.90
Purlin	150S76-250 (+16)	0.94	150S76-250 (+16)	0.94
Girt	200S76-250 (+16)	0.87	200S76-250 (+16)	0.87
Eave strut	150S51-250 (+16)	0.72	150S51-250 (+16)	0.65
Wall bracing	100S51-150	0.46	100S51-150	0.46
Roof bracing	100S51-150	0.59	100S51-150	0.59
Bracing for Rafter against lateral buckling (in the side wall direction)	100S51-150	0.36	100S51-150	0.36
Bracing for Rafter against lateral buckling (in the end wall direction)	150S76-250 (+16)	0.81	//	//

4.3 Financial evaluation

The financial evaluation of any steel building construction is a crucial criterion in the economic aspect. The construction of a CFS building, including the additional supply fees, is estimated to be more expensive than the construction of an HRS building. For instance, the difference is estimated at around 25%. Hence, in this present work, a comparative price study in terms of percentage is established by weighting the total weight of the CFS building by 1.25. Thus, the Unit Price (UP) per kilogram (Kg) of construction for the two types of buildings becomes similar. Remember that this study concerns only the designed building's elements presented in Table 2 and Table 3, without considering the price of the assembly of steel elements and their realizations fees (Washer, Bolts, and Welding: RBS), which is clearly more important and expensive in the case of a building made of M.1-HRS than in a M.1-CFS building. Table 6 presents the comparative results obtained for the two models M.1-HRS and M.1-CFS, leading to conclude that the realization of a M.1-CFS building is more economical than the M. 1-HRS building by 28%, while both steel structures are designed according to Eurocode 3.

Table 6. Financial evaluation of CFS and HRS buildings' element

Design codes	Models	Weight [Kg]	Unit Price UP/kg	Total price (%)
EC 3: P1-1	M. 1-HRS	57136	PU	x
EC 3: P1-3	M. 1-CFS	32609	1.25*PU	0.72x

5. Conclusion

This present study aimed to compare an industrial building made of Cold Formed Steel versus Hot Rolled Steel elements while the verification of the resistance, structural rigidity and global stability are checked by performing several numerical modeling. First, a numerical model of a HRS structure is developed by SAP2000 software and designed according to Eurocode 3 (EC 3 P1-1). Then, the same structure with CFS elements model is designed according to Eurocode 3 (EC 3: P1-3) and the three methodologies defined by the AISI code (ASD, LRFD and LSD). The obtained numerical results led to a several potential comparisons in terms of weight, cost, average CD criterion. These comparisons can be reviewed as follow:

- According to Eurocode 3, the design of an industrial CFS warehouse is more economical than a HRS warehouse in terms of total weight by 43% and cost by 28%;
- The designed industrial CFS warehouse according to Eurocode 3 (EC 3: P1-3) reveals to be less conservative than the AISI design methodologies;
- The comparison results in terms of average CD ratio obtained by the three design methodologies of the AISI code shows that the ASD methodology is the most conservative approach with a difference of 11% and 26% with the LRFD and the LSD approaches, respectively;

In addition, the CFS structure elements designed according to Eurocode 3: P1-3 have been compared through different structural configuration. A first configuration consists of CFS warehouse with full web columns and rafters. The second configuration has replaced the full web rafters by a truss element. In, the third configuration, both full web columns and rafters are substituted by truss columns and truss rafters. It turns out that the first configuration is more economical by exhibiting that is weight of 25% lighter than the second configuration, and 14% lighter than the third configuration.

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