Comparison of Steel Structural Systems in a Manufacturing Unit in Barreiro

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Abstract

The aim of this thesis is the comparison between two steel structural systems typically used in industrial buildings: the portal frame and the rigid framed truss.

This comparison is applied to the specific case of a single-storey manufacturing unit with five bays and several requirements relative to the minimum working areas and heights. So, the goal is to achieve the most economical system, based on the evaluation of the total weights, assuming that both of them have equal unit costs.

For that purpose, each solution is designed according to the Eurocodes, followed by the calculation of the total and partial weights of the elements obtained. In a first phase, the classification and quantification of the relevant actions is made. In a second phase, the load combinations and limit states are defined. Finally, the structural models are introduced in the computer program SAP2000, which provides the results needed to perform the safety verifications and design of the elements.

The results obtained revealed that the rigid framed truss is the most economical solution, achieving a mainframe with 17% less weight than the portal frame solution, leading to a total weight 7% lower. The possibility of increasing the stiffness and resistance of trusses with a low increase in weight allows a more efficient rigid frame behavior, resulting in less expensive columns and beams.

The results have also showed that the adoption of RHS columns is more suitable, representing 80% of the weight of H columns.

Keywords: Steel structural systems, Industrial Buildings, Portal Frame, Rigid Framed Truss, Eurocodes.

1. Introduction

An industrial building is a space dedicated to the industrial activity, commonly used to store and produce materials. So, its design is mainly dominated by economic and functional issues, being the visual and aesthetic aspects less relevant [1]. Usually, clients expect these buildings to have big open working areas and heights, and also to be quickly erected, in order to start the labor activity as soon as possible [2].

Because of that, steel is most often chosen for the frame of these type of buildings [2]. Its off-site prefabrication leads to a modular and quickly erection, with reduced site activities, and its high strength to weight ratio allows the design of large span structures and heavy load carrying capacity [1], with reduced, thus economic, elements.

However, steel characteristics provide a wide range of structural possible systems, which means that it is very important to choose the most adequate, in order to obtain the best economy. In other words, there are several suitable solutions that meet the functional requirements of a determined industrial building so, the problem is to find the most economic one. The better economy is achieved by selecting an efficient structural system, more than by performing a highly sophisticated design of the structural elements [3]. Moreover, design offices frequently have tight deadlines for project completion, ending up by using the structural system they are more familiarized with, which does not mean that it is the best solution.

Previous studies on this topic point to four possible basic types of structural systems to apply in industrial buildings, indicating the preference of some instead of others based on the span, the spacing between consecutive frames, the loads to be carried and even other aspects. The types mentioned are [1]: the rigid frames (including portal frames and rigid framed trusses), the pinned frames, the cable-supported roofs and the arched roofs. The last two types are less adopted, due to the specific conditions that have to be fulfilled to make them competitive and, as stated in SECHALO [1], rigid frame solutions are much more efficient on carrying the imposed loads than a pinned frame solution. Therefore, in a current industrial building, there are two main structural systems to evaluate: the portal frame and the rigid framed truss.

Some authors [1] consider the portal frame as the most economic for spans between 25 m and 35 m, for moderate loads, referring the rigid framed truss as the most efficient for spans between 50 m and 100 m and/or for carrying heavy loads. Although they do not define explicitly what heavy and moderate loads mean or what is the spacing between frames. Other authors [2] go further, presenting a charter as shown bellow, where they compare the bare frame weights of portal frames with rigid framed trusses, in function of the span and the spacing between frames. However, they build the comparison for only two values of frame spacing, not defining the loads applied to the structure.

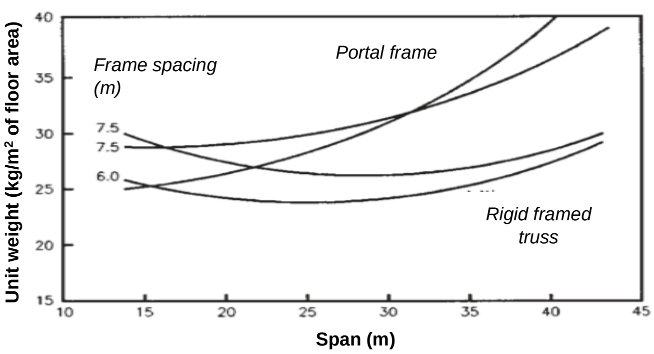


Figure 1 – Comparison of the bare frame weights for portal frames and rigid framed trusses. Adapted from [2]

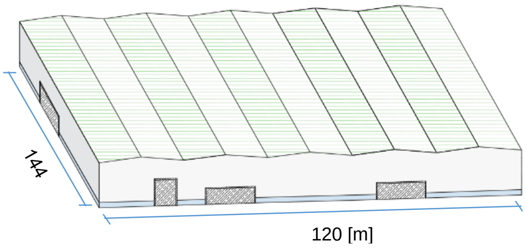
This means that, for a specific industrial building with particular requirements in terms of span, frame spacing, imposed loads, among others, it is not clear which would be the best structural system to adopt. So, the purpose of this paper is to design each one of these two structural systems for a manufacturing unit with specific requirements, in order to show and justify the most economical solution.

2. Methods

2.1 Description of the study object

The manufacturing unit studied, placed at an altitude of 15 m in Barreiro, has the following requirements: the total useful area must be 120x144m^2, formed by a set of 5 bays, each one with an open area of 24x144m^2; the staff and material passage between the bays must be guaranteed, with a frame spacing of 8 m; the minimum height of 11,5 m has to be respected in all the building interior; and the last two bays must have two cranes for equipment and material transportation - the first one with a maximum capacity of 20 ton and the second one with a maximum capacity of 40 ton.

Therefore, the structure to design is a multi-bay single-storey rigid frame, with 5 bays of 24 m and a frame spacing of 8 m. As shown bellow, the clamped columns have 12,75 m and the roof of each bay is duopitched, with a pitch angle of 6º, leading to a maximum height of 14 m.



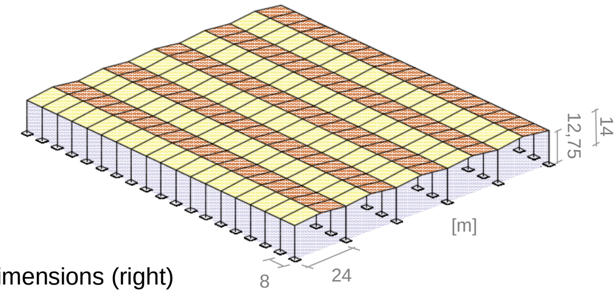


Figure 2 – Manufacturing unit (left) and its structure dimensions (right)

2.2 Eurocode approach

To design both structural solutions, the Eurocode approach described in NP EN 1990 was followed [4]. So, based on the geometry, dimensions and location of the building, the actions were quantified and classified by NP EN 1991 [5-9]. Then the load combinations and limit states to verify according to NP EN 1990 were established, and finally, all elements were designed regarding NP EN 1993 [10] and based on the information provided by the models implemented in SAP2000 [11].

2.2.1 Classification and quantification of actions

The three types of actions are present in the case studied: permanent, variable and accidental. The first type includes the self-weights of the main and secondary frames, with characteristic values between 77 and 78,5 kN/m^3, and the cladding using sandwich panels, with characteristic values of 11,0 kN/m^2 and 10,6 kN/m^2, respectively in the roof and in the facades. Other non-structural elements and connections, not designed, were considered by applying an increment to the ones referred, assuming a value of 5% for all elements, except for the trusses, with 10%.

Regarding the variable actions, we take in consideration: imposed loads (NP EN 1991-1-1 [5]), snow loads (NP EN 1991-1-3 [6]), wind loads (NP EN 1991-1-4 [7]), thermal actions (NP EN 1991-1-5 [8]) and actions induced by cranes (NP EN 1991-3 [9]).

Classified as a roof of category H, the imposed loads take the values of q_k = 0,4 kN/m^2 and Q_k = 1 kN, with the possibility to act independently in any part of the roof. As for the snow loads, the building is in

Zone Z3, which only implies the consideration of drifted and undrifted load arrangements, like shown bellow, in figure 3.

The wind load was calculated using the pressure coefficients method. Located in the velocity Zone B with terrain category I, a peak wind pressure of q_p = 1,751 kN/m^2 was obtained. Therefore, after calculating the resultant of the pressure coefficients, and having a unitary structural coefficient, it was possible to achieve the resultant wind pressures (F_w/m^2) acting on the roof and facades for several design situations. The total number of design situations equals ten and, due to the big dimensions of the building, friction pressures (F_{fr}/m^2) have to be taken in consideration. To a better perception, one design situation correspondent to the wind direction θ=0º is shown in figure 3.

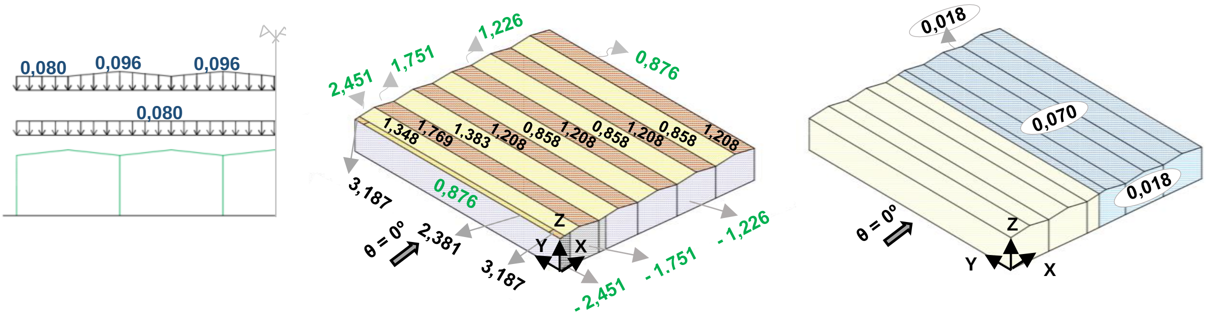


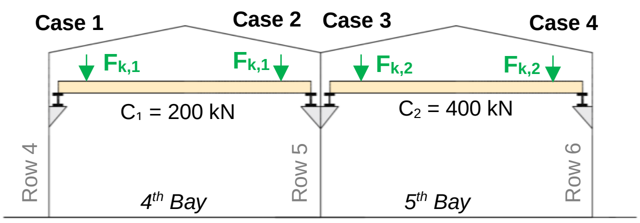
Figure 3 – Characteristic values (kN/m^2) and design situations for snow and wind actions

In what concerns thermal actions, the only component considered was the uniform one, since the sandwich panels include thermal insulation. Located in Zone B, with light panels, the values of ΔT_u (summer) = 18,5 ºC and ΔT_u (winter) = - 6 ºC were considered. As for the actions induced by cranes, were only considered vertical static loads (directly obtained from information provided by the crane supplier in the table 1), since the dynamic and horizontal effects were neglected. The persistent design situations use the service loads, and the cases to evaluate are nine (4 for the runway beams and 5 for the columns), as shown in figure 4.

Finally, regarding the accidental actions, only a simplified case was considered, concerning the situations where the cranes are carrying the maximum capacity, for the same design situations already mentioned. It was assumed that no more actions are applied simultaneously, apart from the permanent loads, of course.

Table 1 – Crane's weight and capacity

Bay	Maximum Capacity	Weight	Service Loads
4th	200 kN	20 kN	110 kN
5th	400 kN	30 kN	220 kN



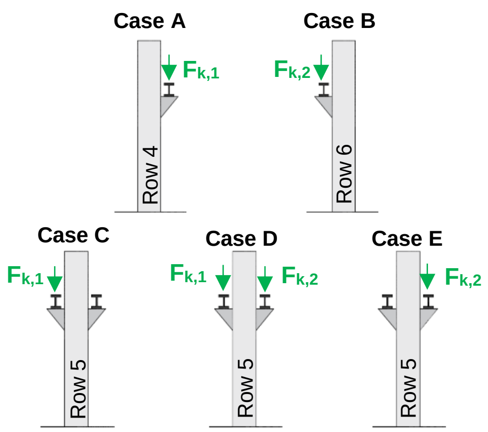


Figure 4 – Design situations for runway beams (left) and columns (right) due to actions induced by cranes.

2.2.2 Load Combinations and limit states

The ultimate limit states evaluated were the structural (STR), related to the collapse and loss of structural stability. Therefore, the safety verifications considered were the ones related to the cross-sectional resistance and buckling resistance of the elements, and the ones concerning the global stability of the structure. Having persistent (1) and accidental (2) design situations, the load combinations in stake were given by:

$$E_d = \sum_{j\geq1} \gamma_{G,j}\, G_{k,j} + \gamma_{Q,1}\, Q_{k,1} + \sum_{i\geq1} \gamma_{Q,i}\, \Psi_{0,i}\, Q_{k,i} \quad (1)$$

$$E_d = \sum_{j\geq1} G_{k,j} + A_d + (\Psi_{1,1}\ or\ \Psi_{2,1})\, Q_{k,1} + \sum_{i\geq1} \Psi_{2,i}\, Q_{k,i} = \sum_{j\geq1} G_{k,j} + A_d \quad (2)$$

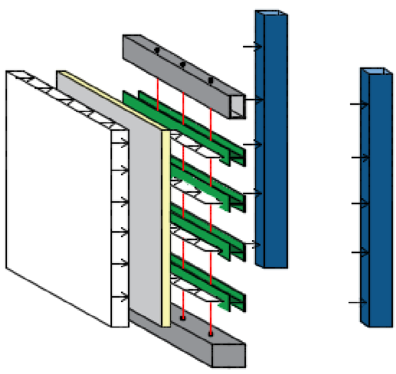
Concerning the serviceability limit state, the horizontal and vertical deflections of the elements were controlled (with the limits of δ_2 =L/250 and δ_{max} = L/200, except for the runway beams, with δ_2 =L/300 and δ_{max} = L/250), as the horizontal displacements of the structure (with the limit of H/150). Lacking an indication from the construction owner, the load combinations evaluated took the format of the characteristic combination (3), given by:

$$E_d = \sum_{j\geq1} G_{k,j} + Q_{k,1} + \sum_{i\geq1} \Psi_{0,i}\, Q_{k,i} \quad (3)$$

2.3 Structure Design

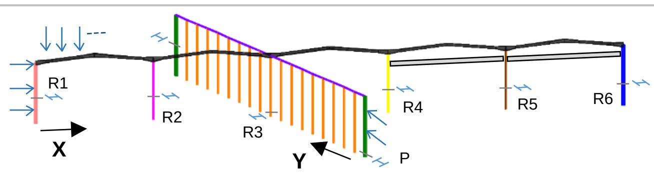
The first step was the implementation of the structural models in SAP2000, assigning the loads previously described and rigid connections between beams and columns and between columns and foundations. In this stage, it was crucial to choose the right profiles to use in each element, in accordance with the chosen structural concept. This structural concept and the elements used are presented and explained in the following figures.

Load transmission system



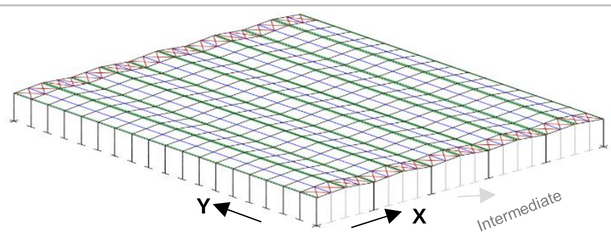
All external loads (on the roof and both facades) are transmitted to the cladding system, then absorbed by the purlins and finally carried by the columns or beams. A clamping system was adopted using RHS and pre-stressed cables to improve the purlins out-of-plane behavior.

Rigid frame behavior in both directions



All columns have the major direction of inertia in X, except the ones on frontal facade. Two profile solutions were evaluated: one in RHS and other in HEB.

Bracing system



The span value is large in the X direction, reason why intermediate facade columns are adopted and also a bracing system, connecting the purlins and main frame with cables.

Figure 5 – Structural concept and elements used

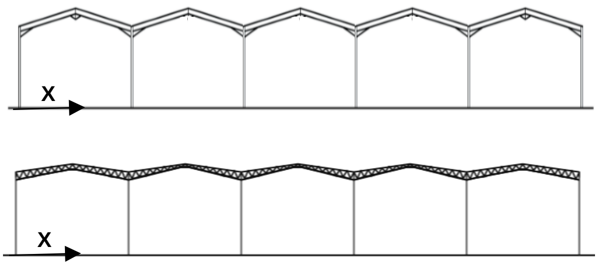
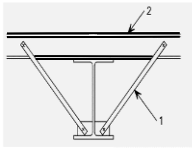
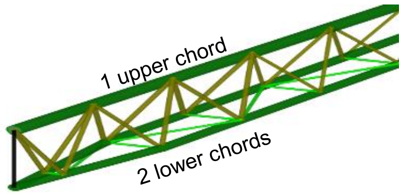
In-plane behavior	**IPE500 beam with restraints**
 In both systems, the major stresses in the beams are introduced by vertical loading, Thus, they work mainly in the major direction of inertia: in the portal frame solution, the principal stress is bending, with a force diagram similar to the case of a continuously supported beam; in the rigid framed truss solution, the main stresses are compression/tension, with a similar force diagram. Since the maximum stresses occur over the supports, those zones are provided with higher beam depths: in the portal frame solution, eaves haunches were used; in the truss solution, the distance between chords was increased.	 A restraint system using CHS profiles (1) connected to the purlins (2) was adopted to improve the out-of-plane behavior of the beam. **Spatial truss using CHS**  It was decided to use spatial trusses, formed by CHS profiles, in order to accommodate the out-of-plane instability.

Figure 6 – Structural concept and elements used

With the information extracted from the computer program, it was possible to safely design each member size, in accordance to NP EN 1993 [10], and finally calculate the partial and total weights of the structure.

3. Results and Discussion

3.1 HEB vs RHS Columns

In the following table are listed the minimum RHS and HEB profiles that verify structural safety according to NP EN 1993, for each row of columns.

Table 2 – Comparison of RHS columns with HEB columns in both structural systems

Row	Portal Frame				Rigid Framed Truss			
	RHS Columns		HEB Columns		RHS Columns		HEB Columns	
	Profile	M (kg/m)	Profile	M (kg/m)	Profile	M (kg/m)	Profile	M (kg/m)
R1	600x400x12	181,0	500	187,0	600x400x12	181,0	450	171,0
R2	300x300x8	72,8	280	103,0	300x200x8	60,3	260	93,0
R3	300x300x8	72,8	260	93,0	300x200x6,3	47,9	240	83,2
R4	300x300x8	72,8	280	103,0	300x200x8	60,3	260	93,0
R5	300x300x10	90,2	360	142,0	300x200x14,2	103	360	142,0
R6	600x400x12	181,0	500	187,0	600x400x12	181,0	450	171,0
P	750x500x16	301,0	900x391	391,0	750x500x16	301,0	900x391	391,0
Intermediate	500x200x10	106,0	340	134,0	500x200x10	106,0	340	134,0
Total	244 950 (kg)		303 889 (kg)		235 210 (kg)		289 824 (kg)	

Two main conclusions can be reached. Firstly, the rigid framed truss system achieves lighter, thus less expensive, columns. In fact, regarding both RHS and the HEB columns, the solutions obtained are about

5% lighter. Secondly, the RHS columns lead to more economical solutions in both structural systems, representing about 80% of the HEB columns' weight. Let us try to explain this last observation.

On one hand, there is a wide range of available commercial RHS profiles, in which length, width and thickness vary. Therefore, they are suitable to be adapted to the needs of resistance and stiffness in both directions of inertia. In HEB profiles, on the other hand, the range of commercial solutions is smaller and there is always a dominant direction of inertia. So, since the structure needs to ensure a rigid frame behavior in both directions (because the vertical bracing adoption is impossible due to the initial open space requirements), it is clear that the RHS columns will induce a better structural performance.

Furthermore, since it is not possible to provide intermediate restraints in the columns, the solution with HEB columns clearly loses competitiveness. This type of profiles, unlike RHS ones, are affected by torsional buckling problems and tend to be rather slender in the minor direction of inertia, especially in tall columns. Thus, a safe design of these elements leads to heavier columns in order to resist to the same applied loads. In fact, consulting table 2, HEB columns have more weight than RHS for the majority of the rows.

3.2 Portal Frame vs Rigid Framed Truss

The table presented below shows the partial and total weights of the two structural solutions obtained using the most economical columns – RHS. All the values were obtained by the multiplication of the unit weights by the number of linear/square meters in which they are used (depending on the element). Figure 7 gathers the information of table 3, providing the weight percentages of the different parts of each structure relative to the total weight.

Table 3 – Weight comparison between the portal frame and the rigid framed truss solutions

Part	Elements	Portal Frame		Rigid Framed Truss	
		Weight (kg)	**kg/m^2**	**Weight (kg)**	**kg/m^2**
Roof	Cladding	474 303	27,5	474 303	27,5
	Purlins				
	Clamping				
	Bracing				
Facades	Cladding		9,1		9,1
	Purlins				
	Clamping	157 679		157 679	
Main Frame	Beams in X	207 131	16,5	150 119	11,9
	Beams Restraint	2 429		-	
	Gable beams in X	10 634		12 311	
	Beams in Y			32 740	
	Gable beams in Y	64 865		9 836	
	RHS Columns	244 950	14,2	235 210	13,6
Crane and Support System	Runway Beams and Cranes Bay 4	33 752	4,6	33 752	4,6
	Runway Beams and Cranes Bay 5	45 336		45 336	
Total		1 241 079	71,8	1 151 286	66,6

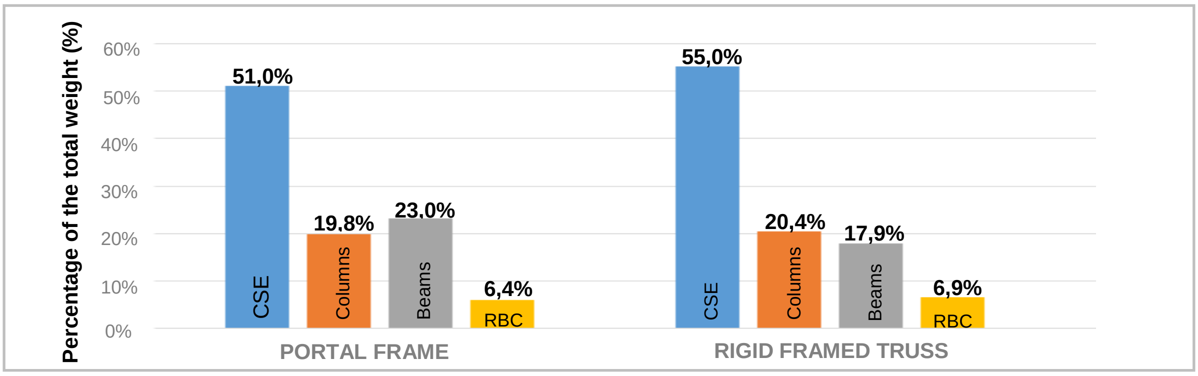


Figure 7 - Weight percentages of the different parts of each structure relative to the total weight

So, through table 3, it is evident that the rigid framed truss solution is the most economical for the manufacturing unit in study, having a total weight of 1 151 286 kg (66,6 kg/m^2 of roof area), which represents about 93% of the portal frame solution weight (with 1 241 079 kg, 71,8 kg/m^2 of roof area).

Recalling SECHALO [1], the conclusions about which would be the most economical system were inaccurate at first. This author pointed to a portal frame solution, for span lengths like the one in study and for moderate loads. Although, it seems that the interior open space requirements and the high wind loads induce a rigid framed truss as more efficient.

Now, recalling Owens and Davison [2], the conclusion about which is the most economical system matches, but the weight values obtained seem to be quite different. In fact, looking at the figure 1, for a span length of 24 m and a frame spacing of 7,5 m (close to 8 m), they obtain a rigid framed truss (with 26,5 kg/m^2 of floor area) 12% lighter than a portal frame solution (with 29,5 kg/m2).

At this point, several aspects must be taken in consideration. In the first place, the values presented by these authors consider only the bare frame weight of the structure. In that case, the results obtained in this thesis would lead to 45,4 kg/m^2 in the rigid framed truss, 10% lighter than the portal frame solution, with 50,6 kg/m^2.

In the second place, though not referring specific characteristics, Owens and Davison evaluation is based in current industrial buildings. Thus, do not cover particular cases where there is the presence of overhead cranes and strong wind effects, like the one in study. Notice that a typical value of the peak wind pressure is about 1 kN/m^2, and the manufacturing unit is submitted to a much larger value of 1,751 kN/m^2, which clearly suggests the need of a heavier structure.

Finally, it should be highlighted that the frame spacing of 8 m is larger than the 7,5 m taken by these authors, introducing bigger loads in the main frame, also suggesting the need of a heavier structure.

Regarding figure 7, it is possible to conclude that the percentages associated to the weight of each part of the structure relative to the total weight are similar in both structural systems, which reflects the identical structural behavior of both of them: the rigid frame behavior.

Another important conclusion is related to the weights of the main frames. In fact, the main frame of the rigid framed truss represents 17% of the portal main frame weight. As it can be observed, the cladding and the secondary structure, equal in both systems, represent more than 50% of the total weights, hiding the true difference between the two systems when comparing the total weights.

At last, it should be noted that the columns represent about 50% of the main frame weight in both cases, a value that could be reduced if there was the possibility to restraint them along the height and/or use a vertical bracing system. In fact, the interior columns' design tends to be conditioned by the flexural buckling (1) and by the horizontal displacements in Y direction, not fulfilling the cross-sectional resistance (2). For example, for the row 5, in the rigid framed truss:

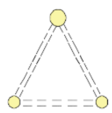
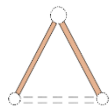
Table 4 – Safety verifications for the RHS columns in row 5 of the rigid framed truss

Row	(1)	(2)	$u^{máx(Y)}$
R5 [RHS 300 x 300 x 10]	$0{,}44 \leq 1$	$0{,}75 \leq 1$	$7{,}9 \leq 8{,}5$

3.3 IPE Beam vs Spatial Truss

The main forces applied on the IPE beam are the bending and shear. Similarly to the truss, bending is mostly absorbed by the compression and tension on the flanges (the chords, in trusses) and shear is mostly absorbed by compression of the webs (the diagonals, in trusses). Thus, to understand the economy achieved with the rigid framed truss in comparison with the portal frame system, it is important to compare these parts of the cross-sections, as presented in the following table 5. Of course, the better economy is achieved for the cross-section in which the geometry and weights allow fully usage of the resistant capacity simultaneously.

Table 5 – Weight comparison of the different parts of the cross-sections between the two structural systems

Elements	**Portal Frame** ($\delta_{2,v}^{max}$ = +9,4 cm) $K_x = 3{,}3 \times 10^7$ (kN/mm) | $K_y = 4{,}2 \times 10^6$ (kN/mm)				**Rigid Framed Truss** ($\delta_{2,v}^{max}$ = +8,5 cm) $K_x = 6{,}8 \times 10^9$ (kN/mm) | $K_y = 2{,}7 \times 10^8$ (kN/mm)			
	Flanges		**Webs**		**Chords**		**Diagonals**	
	Figure	M (kg/m)	Figure	M (kg/m)	Figure	M (kg/m)	Figure	M (kg/m)
Beams in X		50,0		46,7		52,0		10,6
Beams in Y		55,3		16,2		37,9		10,1

Comparing the webs with the diagonals, it can be easily concluded that the adoption of the last ones is much more adequate. Indeed, the webs resistance is not being fully used once it is possible to achieve diagonals with lower weight, withstanding approximately the same loads.

Now, comparing the chords with the flanges, the difference between weights in the X direction is almost inexistent, but in the Y direction it is obtained a lighter solution. However, in either case, the truss solution is more resistant and rigid (see table, where K is bigger and $\delta_{2,v}^{max}$ is smaller), obtaining a better performance. Notice that the stiffness and strength of the solution also depends on the distance between chords/flanges, bigger in the truss case.

Finally, it is important to observe that a better behavior of the trusses leads to a better global performance, the main reason for saving 5% in the columns' weight (see table 2).

4. Conclusions

The purpose of this paper was to design two different structural solutions – a portal frame and a rigid framed truss - for a manufacturing unit with certain initial requirements, in order to compare them in an economic and functional way, providing the best solution. In this way, the objective was fulfilled, achieving the main conclusions bellow:

- The rigid framed truss is the most economical solution, representing about 93% of the portal frame cost, if assumed equal unit costs;
- The largest saving was achieved at the main frame level, reaching 17% less weight in the rigid framed truss than in the portal frame, which was due to the main reasons:
  - Increasing the distance between chords leads to an increase in the stiffness and bending strength of trusses, without practically changing its weight;
  - Using a "web with openings", formed by diagonals, it is possible to adjust the amount of material used to the shear forces applied;
  - The trusses obtained are stiffer than the IPE beams, inducing better rigid frame performance and allowing the use of lighter columns (about 5%). In the rigid framed truss, the columns designed to the ultimate limit states also verify the serviceability limit states, while in the portal frame the serviceability limit state is conditioning in most of the columns;
- The variety of available commercial RHS profiles coupled with the fact that this type of profiles do not have torsional buckling problems gives them a clear competitive advantage compared to H profiles, resulting in columns with 20% less weight.
- The cladding and the secondary structure represent the largest share of the final cost, once its weight composes more than 50% of the total weight.

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