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Design of lightweight mansard portal frames

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Abstract. Single-storey industrial buildings are one of the most often type of structures built among various skeletal framed steel constructions. These metallic buildings offer an exceptional opportunity to minimise the material employed, contributing to a more sustainable construction. In particular, the mansard portal frame is a typology made up of broken beams that involves different lengths and discontinuous slopes. This study aims the weight reduction of the standard mansard portal frame with design purposes by means of varying four parameters: the kink position, the eaves-apex slope, the span and the columns height. In this work, we suggest some guidelines that can improve the economical competitive capabilities of their structural design. In all the cases analysed, the joints of the portal frame are placed over the theoretical non-funicular shape to uniform loads. This allows reducing the bending moment and the shear force, but increasing the axial force. In addition, the performance of mansard and typical pitched portal frames submitted to the same boundary conditions is compared in terms of efficiency in the use of steel. In the large majority of the cases, mansard typologies are lighter than the common pitched frames and, hence, more economical.

Keywords: steel structures; structural design; structural steel; limit state design; hot-rolled steel members; mansard portal frame; kink joint

1. Introduction

The most common type of agro-industrial constructions is a one-storey warehouse with rectangular base, which provides a space protected from weather exposure to carry out activities of production or storage (Davison and Owens 2008). In particular steel portal frames are the most employed typology in these constructions, being close to 90% of all single-storey buildings in countries like UK (McKinstray *et al.* 2015). The total cost of these steel framework structures predominately depends on their weight. Therefore, minimizing the use of raw metal becomes essential in large warehouses for saving resources (Sarma and Adeli 2000, Gurung and Mahendran 2002, Moller *et al.* 2009) in order to sustainably maintain a competitive position (Flick and Fliegel 2013). Nowadays engineers are demanded to aim economical designs, but fulfilling the structural requirements looking for an optimum of the objective function or the "perfect design" (Kravanja and Žula 2010, Mosquera and Gargoum 2014). Weight reduction not only entails a diminution of the expenses, but also it normally decreases the energy and construction consumptions during the assembly. Therefore, the improvement of the design methods for light but still functional portal frames has become one of the recurring topics in the field of steel structures (Hradil *et al.* 2010, Dai *et al.* 2015).

According to the roof geometry, two different types of steel portal frames can be distinguished. In one hand, pitched roof portal frames are used in the common practice due to their cost effectively and versatility for a wide range of spans (Morris and Plum 1988, Artar and Daloglu 2015a). The optimization of this type of steel structures has been frequently investigated in the past (Kravanja *et al.* 2013) and many optimal analysis have been done in the recent years (Phan *et al.* 2013, Artar and Daloglu 2015b, Gholizadeh and Poorhoseini 2015). In the other hand, mansard portal frames have been less studied in terms of best design (Morales-Rodríguez 2015). They are very efficient for enclosing large volumes (Salter *et al.* 2004) and, therefore, they are often employed in warehouses with agricultural purposes. For example, the gambrel roof form is a shape associated with the traditional North American barn, which has retained its popularity among farmers and builders due to its efficient design (Jackson and Masse 1992).

A mansard portal frame consists of four beams, usually symmetrically placed with respect to the middle plane, with two different slopes (Fig. 1). The position of the kink (beam to beam joints) of the roof coincides with the shape of either a parabolic arch or a circular arch. Whenever the arch is subjected to uniform loading the most efficient proposal is the parabolic shape, because the lines of thrust (compression) would be located within the joint cross

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section (Lawson and Trebilcock 2004). With this purpose, traditionally mansard portal frames follow a parabolic arch in order to reduce as much as possible the flexural stresses and, so that, the polygonal structure defined should be primarily subjected to compression. Nevertheless, in case of asymmetric loading or deviation of the arch from the ideal shape, the joints may have to resist additional bending moments and shear forces which should be specifically study.

In addition, the aesthetic appearance of engineering structures is a subject of growing interest (Jorquera Lucerga and Manterola Armisen 2012). Nowadays the society demands not only a functional result, but also attractive and economically feasible. In this context, some authors find the mansard portal frames which follow a parabolic shape more aesthetically pleasing than other solutions (Jackson and Turnbull 1979).

In this work mansard portal frames with a roof of four straight beams are studied, whose joints are located over the shape of a parabolic arch (López-Perales 2004). In order to facilitate their pre-dimensioning, the influence of the kink position, the eaves-apex slope, the span and the column height on the total weight of the structure is reviewed by means of a set of parametric calculations. Recommendations following the standards are included for design purposes. In terms of steel utilisation, the results are compared with those obtained with double pitched roof portal frames submitted to the same boundary conditions.

2. Portal frames definitions

A series of industrial buildings 84 m long are analysed, considering a distance between adjacent portal frames equal to 6 m (Hernandez *et al.* 2005, Phan *et al.* 2013). Then, along the length there are a total of 15 portal frames joined by means of purlins running continuously all over the structure. This study is focused in the structural response of the intermediate portal frames. Two different roof geometries are evaluated, mansard (Fig. 1, solid line) and double-pitched portal frames (Fig. 1, dashed line), with similar general dimensions for comparing their structural response under the same boundary conditions.

In the case of the mansard geometry, each portal frame is constructed from two columns and four beams in the roof, the bottom beam (B_b) and the top beam (B_t). The kink joint between the B_b-beam and the B_t-beam is called henceforth "joint B_b-B_t". Fig. 1 shows the general dimensions considered, in which l is the frame span; h_s is the half span; h_c is the column height; f is the rise height, s_f is the slope between the eaves level and the apex, B_b is length of B_b-beam, B_t is the length of the B_t-beam and B is the beam length for the double-pitched portal frame. The column bases are assumed to be fixed.

The beam to beam cross sections of the kink joint could become the most critical in case of the presence of maximum bending moments. In order to diminish as much as possible the bending moments and shear forces in the joint B_b-B_t and to increase the compressive axial force, the recommended position of the kink joint is placed over the parabolic arch (Fig. 2). The funicular shape of the arch for uniformly distributed loads is described by a quadratic function (Eq. (1)).

$$y = 4f\left(1-\frac{x}{l}\right)\frac{x}{l} \tag{1}$$

The x-position of the joint B_b-B_t is defined in function of the half-span dimension. The parameter a is chosen to vary between 0.2 and 0.6 at intervals of 0.1 (Fig. 2).Values below $0.2h_s$ and above $0.6h_s$ have been rejected, because they would lead to short B_b-beams or roof slopes in the B_t-beams below 8%. Short beams either in the bottom or the top of the roof would generate geometries similar to the double pitched portal frame, while low slopes could derive in snow and water accumulation. The vast majority of portal frames achieve spans of up to 50 m (Salter *et al.* 2004) using hot-rolled sections in the columns and the rafter members in the roof. Besides, inside the context of agricultural applications, large volumes are recommended. Therefore, the industrial buildings proposed have spans of 30 m, 40 m and 50 m and three possible columns heights of 5 m, 6 m and 7 m are taken into account. As regards the design of the arch in which the polygonal shape is inscribed, Torroja (1962) adopted rise to span ratios (f/l) between 1/5 and 1/7. Regalado (1999) increased this interval to 1/8, because he considered that this is the most visually satisfying relationship. Besides, the second order effects increase considerably when the rise to span ratio is below 1/10,

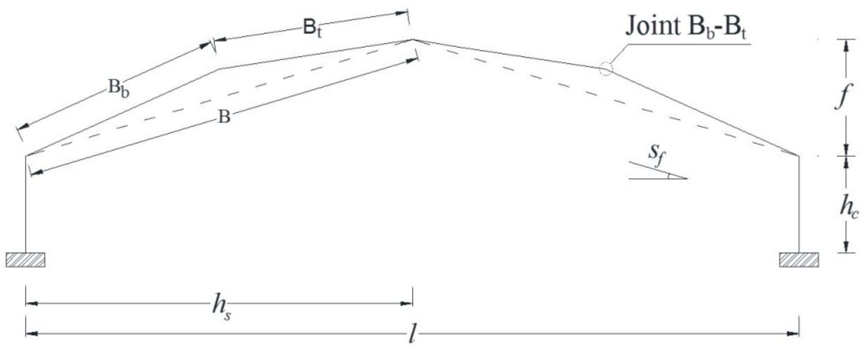

Fig. 1 Portal frames geometry and main dimensions; Mansard roof geometry (solid line); Double-pitched roof geometry (dashed line)

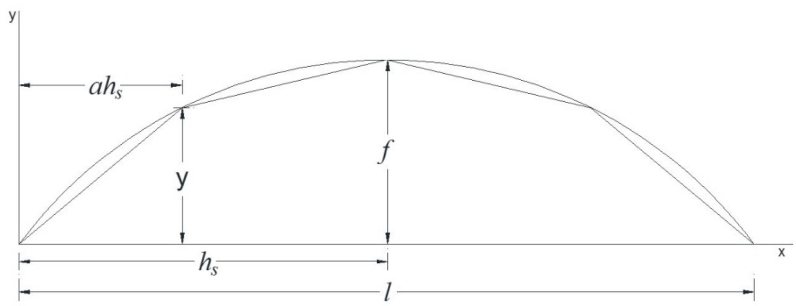

Fig. 2 Coordinates definition; Position of the kink joint B_b-B_t

Table 1 Summary of the parameters and their values

Span (l)	30 m		40 m		50 m
Eaves-apex slope (s_f)	20%		25%		30%
Column height (h_c)	5 m		6 m		7 m
*Kink joint B_b-B_t x-position (ah_s)	$0.2h_s$	$0.3h_s$	$0.4h_s$	$0.5h_s$	$0.6h_s$

* Only for the mansard portal frames

especially in fixed and relative rigid arches (López-Perales 2004). So that, three different f/l ratios equal to 1/7, 1/8 and 1/10 are considered, which correspond to eaves-apex slopes of 30%, 25% and 20% respectively. Table 1 summarises the values of the geometrical parameters which are analysed in this work.

The structural elements of the portal frames are proposed to be built up from European hot rolled double-T standard sections. H-beam are generally utilised in vertical columns of industrial buildings due to their good torsional response, while I-beams are proposed as the lighter solution in parts working mainly under flexural loading. Therefore, columns consisting of HEB sections and IPE-beams in the roof are proposed for the double pitched portal frames. In relation to the auxiliary members, the studied double-pitched portal frames do have eaves haunches but no auxiliary members are considered in the mansard portal frames. It is adopted the typical length of the eaves haunch for portal frames, 20% of the length of the rafter, that corresponds with approximately 10% of the frame span (Salter *et al.* 2004, Phan *et al.* 2013). The eaves haunches are assumed to be fabricated from the same section size as those of the rafter. Meanwhile, the combinations proposed in the mansard portal frames are depicted in Table 2. With regard to the type of joints, the hypothesis that the rigid joints will be good executed is established. Consequently, they are supposed to be capable of withstanding the stresses and fulfilling the intended level of safety, serviceability and durability.

Concerning the material properties, standard steel of grade S275 is adopted due to its extended used in construction (Gozalvez *et al.* 2014). Only the elastic properties are taken into account because one of the design conditions is that the material should not develop stresses over the elastic limit of the material. Table 3 lists the steel properties, where f_y is the yield strength, E is the Young's

Table 2 Combinations of bottom- and top-beams in mansard portal frames

Beam sections	B_b	B_t
Type 1 (IPE/IPE)	IPE	IPE
Type 2 (HEA/IPE)	HEA	IPE
Type 3 (HEA/HEA)	HEA	HEA

Table 3 Elastic properties of the steel S275 (CTE 2006, EAE 2012)

Yield strength (f_y)	275 N/mm^2
Young's modulus (E)	210000 N/mm^2
Density (ρ)	7850 kg/m^3
Shear modulus (G)	81000 N/mm^2
Poisson´s ratio (ν)	0.3

modulus, ρ is the density, G is the shear modulus and ν is the Poisson's ratio.

To name the different portal frames that are analysed, the following nomenclature is adopted. The first two characters are PF or MF, indicating that the portal frame is a pitched portal frame (PF) or a mansard portal frame (MF). Afterwards, a series of six digits is detailed; the first two numbers show the span of the frame (30 m, 40 m or 50 m), the next two indicate the eaves-apex slope of the roof (20, 25 or 30%) and the last two digits represent the columns height (05, 06 and 07 for the heights of 5 m, 6 m and 7 m respectively). In addition, the nomenclature of the mansard portal frames has an extra label separated from the previous digits by an underscore, indicating the abscissa position of the joint B_b-B_t in relation to the half-span. This means that 02, 03, 04, 05 or 06 is added for making reference to portal frames with the kink joint at 0.2, 0.3, 0.4, 0.5 or 0.6 times the half span (ah_s).

3. Limit state design

The structure is required to satisfy the equilibrium, rigidity, strength and stability criteria. On the one hand the Ultimate Limit State (ULS) condition is reviewed in order to assure the strength and stability under design loads. On the other hand, Serviceably Limit States (SLS) have to be checked to fulfil the functionality, comfort, durability or appearance requirements. The dimensioning of the steel members is performed in accordance with the Spanish Technical Building Code (CTE 2006) for the conditions of the ultimate and the serviceability limit states. In addition, three other regulations related to metal structures such as the Eurocode Structural Steel (Eurocode 3 2005), the Structural Steel Instruction (EAE 2012) and Steel Construction Institute (SCI 2010) are taken into account, as well as two updated documents of the CTE 2006 which are the CTE DB SE-A (2007) and the CTE DB SE-AE (2009).

It is worth to highline that during the analysis, the limitations imposed on the vertical displacements have been chosen to be more stringent than the conditions given by the normative (SCI 2010) in order to avoid excessive deformations in the structure. These deflection constrains have provoked that the Serviceable Limit State would control the frame election and, so that, a purely elastic design could be applied (McKinstray *et al.* 2016). As far as the buckling verifications are concerned, the buckling lengths of the columns and beams are determined by means of the effective length factor K (Fig. 3). Meanwhile, the guidelines of the CTE DB SE-A (2007) state that the lateral torsional buckling of the beam shall not be verified whenever its compressed wing is restrained at distances less than 40 times the minimum radius of gyration. In all cases simulated the required bracing distance that should permit not to consider in the design this stability condition is included between the range of 1.72 m and 1.86 m for the B_b and 1.21 m and 1.86 m for the B_t. This work proposes as a design assumption that the beams are braced at distances

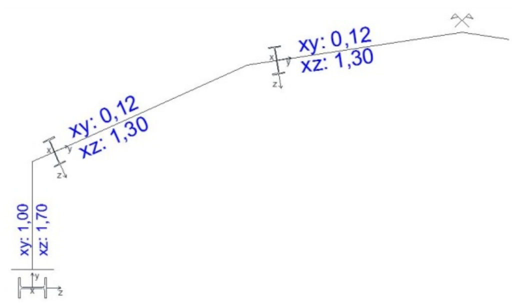

Fig. 3 Effective length factors

below the most limiting one, which is for any case 1.21 m. For this reason and according with the normative, lateral torsional buckling is not necessary to be taken in account because it is assumed as initial hypothesis that the structure is sufficiently braced.

The internal forces and deflections are calculated from the external actions by means of an elastic first-order analysis. As it is described in EAE (2012), the sway imperfections may be ignored in building frames for a certain combination of actions if $H_{Ed}/V_{Ed} \geq 0.15$. The design values H_{Ed} and V_{Ed} are the resultant loads of the total horizontal and vertical reactions, respectively, at the building base. This condition is satisfied in all the cases studied, varying the relation H_{Ed}/V_{Ed} from 0.8 to 2.0. Therefore, were have ignored second-order effects for design purposes because the influence of equivalent global sway imperfections on the structure would be less significant than the consequence of the acting horizontal forces (Lim *et al.* 2005).

Furthermore, though much commercial software carries out structural calculations, a spreadsheet has been developed ad hoc to determine the action of the most unfavourable external loads combination. Nonetheless, due to the big amount of data handled, the final optimized sizing has been obtained with the help of the commercial software CYPE. The Direct Stiffness Method (DSM) is used not only by the software CYPE but also it has been implemented in the spreadsheet developed ad hoc. The DSM develops the matrix analysis of structural systems that can be described as a set of mono-dimensional elements interconnected by nodes. Once the stiffness matrix of the structure is defined, the unknown displacements and forces can then be determined. Mono-dimensional elements (Geometry 1D) have been utilised.

The actions on the structures have been determined according to CTE (2006) in its basic structural safety document "Actions in the building" (CTE DB SE-AE 2009). The loads considered in the design are detailed in the following subsections:

3.1 Dead loads

The permanent or static loads that have been considered are:

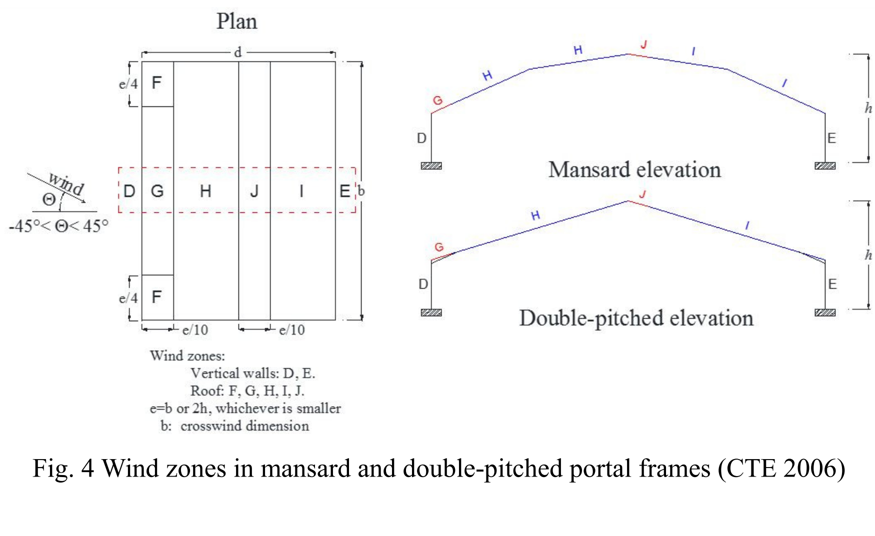

Fig. 4 Wind zones in mansard and double-pitched portal frames (CTE 2006)

- Self-weight of portal frame. It is calculated as the sum of each bar mass, calculated from their density, length and cross section area.
- Weight of fixed service equipments and other permanent machinery (0.15 kN/m^2).
- Roofing material: composite panel (0.15 kN/m^2).
- Weight of purlins and fixed elements (0.12 kN/m). This weight per meter of length is estimated for an IPE 140 cross section of the purlin.

3.2 Variable loads

The non-permanent loads that are taken into account are the use loading and the result of environmental conditions as wind and snow. The following hypotheses are adopted in relation to the industrial building:

- It is placed in the wind zone A (it covers most of the Iberian Peninsula).
- Its environment has a degree III of roughness (uneven or flat rural area with some isolated obstacles such as trees or small buildings).
- It is not included the existence of pressure or suctions since less than 30% openings are considered.

According to these conditions, the CTE (2006) indicates that the wind pressure should be applicable to different zones depicted in Fig. 4 for both mansard and double-pitched geometries. Please notice that we do not have taken into account the wing zones A, B and C defined by the standard in the facade of the buildings. The reason is that this work is focused on the structural response of an intermediate portal frame, considering that appropriate racing is developed all along the industrial building. Therefore, the study assumes as initial hypothesis that there are no longitudinal effects on the intermediate columns and those are only subjected to bending moments in one plane due to the unavoidable lateral wing pressure. Afterwards, a weighted average method to calculate the wind loads on beams located in more than one wind zone is used. Finally, both snow and use overloads are 0.6 kN/m^2 and 0.4 kN/m^2, respectively, following the specifications given by the normative.

4. Results and discussion

A total of 486 different portal frames have been analysed, although only the most representative cases are presented and commented for simplicity reasons. The results obtained after the dimensioning of the beams cross sections are exposed paying special attention to the lighter solutions with economical purposes.

In order to prove that the mansard geometry could be a less weighted design than the classical double-pitched solution, a comparison of the results obtained in both cases with similar dimensions and boundary conditions are shown in Tables 4-6. For each couple mansard/pitched portal frames that are compared, the minimum weight case is taken as a reference for calculating the deviation from the lighter option (Δ). In general terms, the combination of beam sections that lead to lower weight structures is made up of IPE rafters. The exceptions appear in the mansard portal frames MF402007_04, MF502006_05, MF502007_ 03 and MF502507_04 in which the HEA/IPE combination

Table 4 Comparison of results for mansard and pitched portal frames with 30 m of span

Portal frame	Column section	Rafter section		Frame weight (kg)	Δ
MF302005_04	HEB 340	IPE 550	IPE 360	3702	-
PF302005	HEB 360	IPE 450		4019	8,6%
MF302505_06	HEB 360	IPE 500	IPE 270	3589	-
PF302505	HEB 360	IPE 450		4047	12,8%
MF303005_06	HEB 340	IPE 500	IPE 270	3555	-
PF303005	HEB 340	IPE 400		3617	1,8%
MF302006_03	HEB 360	IPE 550	IPE 400	4108	-
PF302006	HEB 400	IPE 450		4464	8,7%
MF302506_04	HEB 360	IPE 550	IPE 360	4100	-
PF302506	HEB 360	IPE 450		4330	5,6%
MF303006_05	HEB 360	IPE 500	IPE 330	3945	-
PF303006	HEB 360	IPE 450		4364	10.6%
MF302007_04	HEB 400	IPE 550	IPE 400	4702	-
PF302007	HEB 400	IPE 450		4775	1,6%
MF302507_05	HEB 360	IPE 550	IPE 360	4532	-
PF302507	HEB 400	IPE 450		4803	6,0%
MF303007_06	HEB 360	IPE 550	IPE 300	4549	-
PF303007	HEB 400	IPE 450		4837	6,3%

*Δ: Percentage of increasement in the weight between the compared structures

Table 5 Comparison of results for mansard and pitched portal frames with 40 m of span

Portal frame	Column section	Rafter section		Frame weight (kg)	Δ
MF402005_05	HEB 550	IPE 600	IPE 400	5885	-
PF402005	HEB 500	IPE 550		6578	11,8%
MF402505_06	HEB 500	IPE 600	IPE 330	5777	-
PF402505	HEB 450	IPE 550		6467	12,0%
MF403005_05	HEB 500	IPE 550	IPE 400	5522	-
PF403005	HEB 450	IPE 550		5588	1,2%
MF402006_04	HEB 700	IPE 600	IPE 450	6818	-
PF402006	HEB 500	IPE 550		6953	2,0%
MF402506_04	HEB 500	IPE 600	IPE 450	6240	-
PF402506	HEB 500	IPE 550		7004	12,2%
MF403006_06	HEB 500	IPE 600	IPE 330	6227	-
PF403006	HEB 500	IPE 550		7065	13,5%
MF402007_04	HEB 500	HEA 550	IPE 500	7616	-
PF402007	HEB 500	IPE 600		8101	6,4%
MF402507_04	HEB 700	IPE 600	IPE 450	7360	-
PF402507	HEB 500	IPE 550		7378	0.3%
MF403007_05	HEB 550	IPE 600	IPE 400	6820	-
PF403007	HEB 500	IPE 550		7439	9,1%

*Δ: Percentage of increasement in the weight between the compared structures

Table 6 Comparison of results for mansard and pitched portal frames with 50 m of span

Portal frame	Column section	Rafter section		Frame weight (kg)	Δ
MF502005_04	HEB 900	IPE 600	IPE 500	8238	-
PF502005	HEB 600	IPE 600[a]		9119	10.7%
MF502505_04	HEB 800	IPE 600	IPE 500	8024	-
PF502505	HEB 600	IPE 600		9042	12,7%
MF503005_05	HEB 700	IPE 600	IPE 450	7723	-
PF503005	HEB 550	IPE 600		9006	16,6%
MF502006_05	HEB 800	HEA 600	IPE 500	9998	3,1%
PF502006	HEB 650	IPE 600[a]		9697	-
MF502506_04	HEB 900	IPE 600	IPE 500	8898	-
PF502506	HEB 650	IPE 600[a]		9619	8,1%
MF503006_04	HEB 800	IPE 600	IPE500	8641	-
PF503006	HEB 600	IPE 600		9555	10.6%
MF502007_03	HEB 700	HEA 650	IPE 600	10700	1,7%
PF502007	HEB 700	IPE 600[b]		10518	-
MF502507_04	HEB 800	HEA 550	IPE 550	10445	2,2%
PF502507	HEB 700	IPE 600[a]		10222	-
MF503007_04	HEB 1000	IPE 600	IPE 500	9888	-
PF503007	HEB 650	IPE 600[a]		10313	4,3%

*Δ: Percentage of increasement in the weight between the compared structures

is the proposed solution, not being the lighter design the last three cases with 50 m of span. As well, whenever the joint B_b-B_t is at $0.2h_s$, only HEA sections (HEA/HEA) are of interest in terms of lightness. The reason is that due to the considerable length of the upper beams (IPE) the limit requirement on the vertical displacements is exceeded. Nevertheless, the position of the kink joint located at $0.2h_s$ never provides the lightest structure. For clarifying this

Table 7 Position of the joint B_b-B_t which leads to the lighter mansard portal frame

Eaves-apex slope (s_f)	20%			25%			30%		
Columns height (h_c)	5 m	6 m	7 m	5 m	6 m	7 m	5 m	6 m	7 m
30 m of span	$0.4h_s$	$0.3h_s$	$0.4h_s$	$0.6h_s$	$0.4h_s$	$0.5h_s$	$0.6h_s$	$0.5h_s$	$0.6h_s$
40 m of span	$0.5h_s$	$0.4h_s$	$0.4h_s$	$0.6h_s$	$0.4h_s$	$0.4h_s$	$0.5h_s$	$0.6h_s$	$0.5h_s$
50 m of span	$0.4h_s$	$0.5h_s$	$0.3h_s$	$0.4h_s$	$0.4h_s$	$0.4h_s$	$0.5h_s$	$0.5h_s$	$0.4h_s$

affirmation, the position of the joint B_b-B_t which leads to the lighter mansard portal frame is depicted in Table 7. It can be observed that the kink located at $0.4h_s$ provides the greatest number of structures with lighter design (44.4% of the cases).

For illustrating the influence of each variable in the structural weight reduction, the following subsections explain the effect of varying the values of the four parameters proposed in Table 1: the kink position (joint B_b-B_t), the span, the eaves-apex slope and the columns height. In each case, the total mass of the portal frame is calculated as the sum of the columns and beams masses. In addition, in the double-pitched portal frames is comprised the mass of the eaves haunches.

4.1 Influence of the span and the position of the joint B_b-B_t

This subsection is focused on the results obtained in mansard portal frames with a fixed eaves-apex slope equal to 20% and a height column of 5 m. Then, Fig. 5 shows the effect on the frame weight of varying the joint B_b-B_t position and the span. As it can be expected, the involved mass is greater when the structure has more span, due to an increase of the lengths and sections of the inclined beams and, consequently, of the columns cross sections.

In this sense a greater impact is observed when the span of the frame increases from 30 m to 40 m (mass values between 49.9% and 66.0% heavier) than when it increases from 40 m to 50 m (mass values between 38.8% and 45.0% heavier). In any case, it should be outlined the clear tendency to minimise the structural weight whenever the joint B_b-B_t position is close to $0.4h_s$.

4.2 Influence of the eaves-apex slope and the position of the joint B_b-B_t

Following the previous discussion, here the attention is centred in structures with the span length and the column height fixed to 40 m and 7 m, respectively. Then, the weight variation is observed regarding the eaves-apex slope and the position of the joint B_b-B_t.

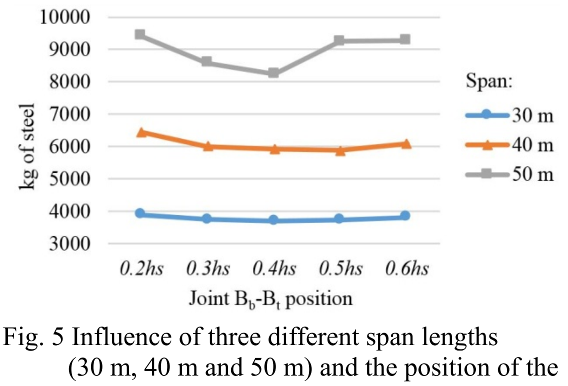

Fig. 5 Influence of three different span lengths (30 m, 40 m and 50 m) and the position of the joint B_b-B_t on the frame weight

Any increasement of the slope leads to a decreasement in weight. The reason is that a reduction of the bending moment and an augmentation of the compressive axial force on the beams is produced, what makes possible a better use of the steel employed. Once again, the tendency depicted in Fig. 6 shows clearly that the lighter structures are those with the joint B_b-B_t x-position close to $0.4h_s$, placed over the guideline of the parabolic arch.

4.3 Influence of the column height and the position of the joint B_b-B_t

The last scenario considers mansard portal frames with 40 m of span and an eaves-apex slope of 20%. Fig. 7 shows the dependency of the total mass involved with the position of the joint B_b-B_t and the three different heights taken into account. The results show that a different columns height is

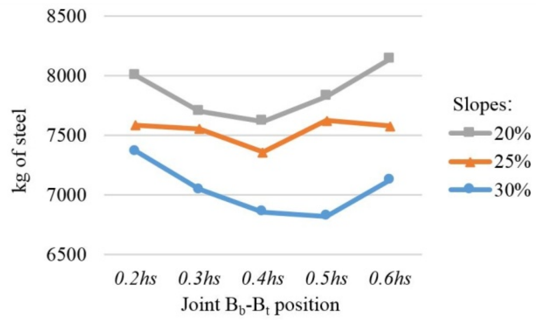

Fig. 6 Influence of three different eaves-apex slopes (20%, 30% and 40%) and the position of the joint B_b-B_t on the frame weight

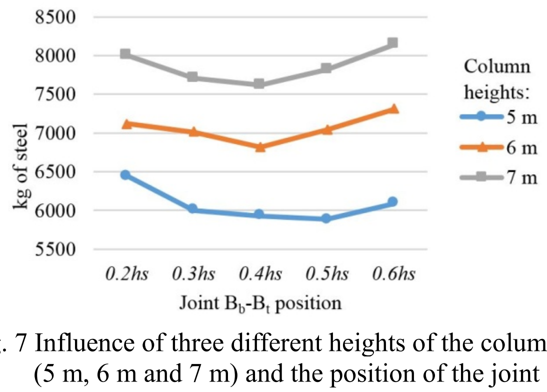

Fig. 7 Influence of three different heights of the columns (5 m, 6 m and 7 m) and the position of the joint B_b-B_t on the frame weight

not a determinant factor for the dimensioning of the roof beams. Therefore, a certain homogeneity on the weight increasement is observed when the columns height changes. A column height of 7 m shows mass values between 7.0% and 17.9% higher than the structures with a column height of 6 m, being these values between the 8.0% and the 21.4% higher than the ones obtained in a portal frames with a column height of 5 m. As in the precedent cases, the minimum weight structures are those in which the joint B_b-B_t position is close to $0.4h_s$. This effect is observed in all the mansard portal frames analysed. Generalising, $0.4h_s$ is a recommendable position of the kink joint in order to obtain a lighter but still functional portal frame, ensuring that the portal frame will be the lightest (within a maximum weight deviation of 5%).

5. Conclusions

The study presents a set of recommendations with the aim of pre-designing mansard portal frames. The considered structures are intermediate portal frames, which are mutually connected to each other by means of purlins. The constraints and the loads are defined based on current regulations. Several geometries are studied being the principal aim to determine the positions of the joints B_b-B_t which leads to lighter structures and the influence of the span, slope and columns height on the results. The main conclusions obtained are:

- The position of the joints B_b-B_t of the mansard portal frame that follows the anti-funicular line corresponding to a distributed load, does not guarantee that the joint works mainly under compression due to the non-uniform distribution of loads, but it decreases the bending moment and shear loads that it develops. Sometimes, this does not imply excess savings when the profiles are dimensioned; however, it conditions its behaviour and simplifies the design of the joint B_b-B_t due to the reduction of the flexural forces.
- In general terms, the most recommended position of the joint between beams is $0.4h_s$. This ensures that the portal frame will be the lightest within a maximum weight deviation of 5%.
- The position of the joint B_b-B_t located at $0.2h_s$ never provides the optimal weight.
- The variable with most influence is the span length, followed by the column height and finally the slope. Unlike the other two, an increasement of slope leads to a decreasement of the weight.
- In the vast majority of the portal frames studied under the same conditions, mansard typologies are lighter than the common pitched frames with haunch rafters. Hence, mansard geometries are more economical from the point of view of the amount of steel employed and they maximise the useful volume inside the industrial building. Nevertheless, it should be noted that the possibility of using bracketed joints could reduce the weight of any of the studied geometries.

Furthermore, the position of the joint B_b-B_t not only affects to the total weight of the frame but also to its structural aesthetic which is a variable difficult to quantify. If the kink joint is placed at $0.2h_s$, then it produces a visual disparity between the two rafter sections. Meanwhile, whenever the kink joint is located at $0.6h_s$, the optical effect of the zenithal zone does not allow a clear perception of the broken line of the mansard roof. More visually pleasant cases could be considered those in which the polygonal lines that define the elevation of the mansard roof fit better with the geometry of the arch. This happens whenever the break joint is located between $0.3h_s$ and $0.5h_s$. Then, taking into account the minimum weight and aesthetic conditions (despite of its subjectivity), both they seem to coincide with the best results at $0.4h_s$. As Khan stated (1980), whenever an industrial building is designed in an efficient, simple and sensitive way the structural solution would reach also visual strength and presence.

Finally, it is worth to remember that this work has been developed under certain initial assumptions that have been imposed, starting from the geographical situation of the industrial building or its boundary conditions and finishing with the ideal execution of the rigid joints or the enough braced beams. These initial hypotheses are needed to be established for being able to develop the parametric studies in which the work is focused and that allow to describe the weight reduction of the mansard portal frame with respect to the pitched portal frames considering identical scenarios. The starting conditions are considered as the reference state for both portal frame geometries, treating the parametric analysis carried out isolating one of the intermediate portal frames as independent of the reference situation.

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