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Impact of joint stiffness on the overall cost of gabled steel frames

Impacto de la rigidez de las uniones en el coste global de pórticos a dos aguas de acero.

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ABSTRACT

This paper studies the influence of joint stiffness value combinations in gabled steel frames. Twelve different joint stiffness values were combined for the column base, knee joint and ridge joint. Ten different geometries were combined for gabled steel frames (8-20 m span, 3.5-10 m column height and 10-20% roof slope). Finally, three different load levels for snow and wind were considered, with constant values of permanent and maintenance loads. A total of 46656 cases were analyzed using specific software for steel structure design, obtaining in each case the total cost. Graphical cost representation was obtained for each joint stiffness value combination. Joints represent an average of 17% of the total cost of gabled steel frame structures. In general, lower cost structures were obtained with low joint rigidity values. It is possible to reduce the total cost by around 18% on average, with appropriate selection of joint stiffness combinations.

Keywords: Joints; gabled steel frames; cost; optimization; stiffness.

RESUMEN

En este artículo se presenta un estudio sobre la influencia de la rigidez de las uniones en pórticos a dos aguas de acero. Se combinan doce valores diferentes de rigidez de placa de anclaje, hombro y cumbrera. Se combinan diez geometrías (8- 20 m de luz, 3.5-10 m de altura de pilares y 10-20% de pendiente de cubierta). Se consideran tres niveles de carga de nieve y viento con valores constantes de carga de mantenimiento. Se analizan un total de 46656 supuestos utilizando un programa informático de cálculo de estructuras para obtener en cada caso su coste total. Se obtiene la representación gráfica del coste para cada combinación de rigideces. En general, los costes más bajos de las estructuras se corresponden con valores bajos de rigidez. Es posible reducir el coste total un 18% de media, con la selección apropiada de la combinación de valores de rigidez de las uniones.

Palabras clave: Uniones; pórticos a dos aguas; coste; optimización; rigidez.

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1. INTRODUCTION

The close relationship between frame response and joint behaviour is clear. This makes it necessary to explicitly consider this phenomenon in frame design phases, to obtain accurate results for the behaviour of the structure as a whole. The idea is still widespread that professional steel structure designs are based mainly on the hypothesis that joint behaviour is theoretically pinned or rigid. Joint behaviour depends on their rigidity. Their extreme values are 0 for pinned and infinite for rigid joints. It is evident that there are many intermediate values. This is roughly the basis of the semi-rigid joint concept, which is included in steel structure regulations and legislation, such as Eurocode 3 (1) or AISC (2). According to Bel Hadj Alí *et al.* (3), engineers still find it hard to include semi-rigid connections in their work plans. This is due to the lack of suitable tools which would make design work easier. This has led some authors to work on for developing practical tools for design. These authors include Jaspart (4), who introduces simplified procedures for connection design under Eurocode 3; Steenhuis *et al.* (5), who propose the use of their tables of calculus for more usual kinds of connections; Bijlaard (6).

Connection behaviour is affected by a large number of parameters, which make them complex to model. This also increases the complexity of structural behaviour as a whole. What is more, other factors as soil reaction and foundation behaviour complicate this issue further (Kanvinde *et al.* (7); Kavoura *et al.* (8)). Joints mainly influence the distribution of bending moment in beams and columns, as well as displacement effects (Sanchez and Martí (9); Chen and Toma (10)). In addition, the influence of joints on the instability and buckling is an important point (Kavoura *et al.* (8); Galambos (11)). These influences are both a handicap and an advantage. It is a handicap due to the complexity of structural behaviour prediction and the negative impact of this uncertainty on design. The advantage is the resulting variability, which means there are many structural solutions, one of which is always the best. This may be considered a source of optimization (Bajwa *et al.* (12)).

Although in the last three decades structural optimization has been extensively studied, contrary to expectations engineers have few tools for frame design for economic optimization (Bel Hadj Ali *et al.* (3)). This kind of design requires accurate model-based methods for cost estimation. Several authors can be cited here, such as Watson *et al.* (13), who develop ideas for the determination of the cost of steel structures; Jarnai and Farkas (14), who present a detailed costs function for welded frames; Xu *et al.* (15), who consider a combined cost of elements and joints and that the cost of each connection is a function of its rotational rigidity; Simoes (16), who considers the cost of connections in the optimization of semi-rigid frames. Advanced structural optimization is now being developed by means of methods such as the genetic algorithm (Kameshki and Saka, (17)) or harmonic algorithm (Saka (18)).

Behind of the economic optimization is the optimization of the structural behaviour. This structural optimization consists of achieving one distribution of internal forces and moments in each element and in the whole frame, which avoid a high stresses concentration in some cross-sections of the elements. These stresses concentration drive to the underutilization of the material.

The aim of this paper is to analyze the influence of joint stiffness on the overall cost of gabled steel frames and to determine the best and worse situations. It is based on previous research works by the authors (19).

2. MATERIAL AND METHODS

Work methodology consists of the study of a representative sample of portal frames under specific load conditions and with different connection rigidity values. This sample is subjected to a design and optimization process to identify the combination of joint rigidities which yield the most economical result. The main points of this methodology are described below.

2.1. General definition of the target buildings

The buildings studied are of a light industrial type, one storey high with a gabled roof and steel structure on a reinforced concrete foundation. Their main characteristics are:

- Length: fixed at 40 m in all cases. This parameter only affects wind load values.
- Spans, column heights and roof slopes: see table 1.
- Space between frames: was fixed at 5 m in all cases.
- Structure definition: this study did not consider extreme or close to extreme frames, and only intermediate portal frames were considered. The frame was composed of two columns and two beams which shape the gabled roof. These beams supported the "Z" or "C" purlins, in S 235 JR steel according to the EN 10025-2:2006 standard (20), with yield strength f_y = 235 N/mm² for nominal thickness t ≤ 40 mm. The columns were connected to the foundation by anchorage plates. The foundation was composed of rectangular, centred and single footings.
- Structure material: HEA profiles (S 275 JR, fy = 275 N/mm², t ≤ 40 mm) were selected for columns and IPE (S 275 JR) for beams. Anchorage plates were designed in S 275 JR steel for plates and B 400 S for anchorage according to the EN 10080:2006 standard (21) with yield strength f_y = 400 N/mm². Reinforced concrete footings were chosen (HA-25 and B 400 S). HA-25 concrete is defined according to Spanish code EHE 08 (22). It is equivalent to Eurocode 2 C25/30 (23), with characteristic value of compression strength of 25 N/mm² obtained in cylindrical specimen (15 cm diameter and 30 cm long) or 30 N/mm² obtained in cubic specimen (15 cm side).
- Roof material: for low and medium loads 0.6 mm thick folded sheet steel was considered, and for high loads 40 mm thick sandwich panel was selected.
- Siding material: 150 mm thick precast concrete honeycomb panel.
- Holes in facades: these are only used to maximize and minimize loads, and the criterion selected is that the weighted average height of the mid-point of the holes is 2/3 of the column height.

- Bracing elements: the buildings are braced by diagonal ties (Saint Andrew crosses) in the end modules; and struts from purlins to beams in order to avoid lateral buckling of the beams.

2.2. Geometric delimitation of the issue

To delimit the problem it is essential to determine the parameters, ranges and cases of the study. This therefore centres on only ten different portal frames which are considered representative of the set of possible cases (Table 1). The scheme of the kind of portal frame considering all its joints semi-rigid is shown in figure 1.

Table 1. Geometry of portal frames

Portal frame type Nº.	**Span (m)**	**Height of column (m)**	**Roof slope (%)**
1	20	10	10
2	20	7	10
3	20	5	10
4	15	10	10
5	15	7	10
6	15	5	10
7	10	7	15
8	10	5	15
9	10	3.5	15
10	8	3.5	20

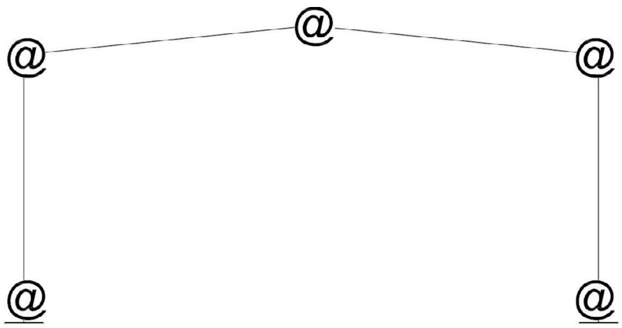

Figure 1. The scheme of the kind of portal frame s tudied with all joints semi-rigid

2.3. Loads considered and their combinations

Three loads levels were considered for variable actions (wind and snow load): low, medium and high, table 2. The Spanish regulation in this field was used to determine each one, more specifically the *Instrucción del Acero Estructural (EAE)* (24) and *Documento Básico de Seguridad Estructural – Acciones en la Edificación (DB-SE-AE)* of the Technical building code *(Código Técnico de la Edificación) [CTE]* (25). The objective was that loads are as real as possible. Table 2 shows the actions considered in this work.

2.4. Stiffness values considered and types of joints

The stiffness values considered for each joint in kN·m/rad were as follows: 0, 5000, 10000, 15000, 20000, 40000, 80000, 100000, 200000, 400000, 800000 and infinite.

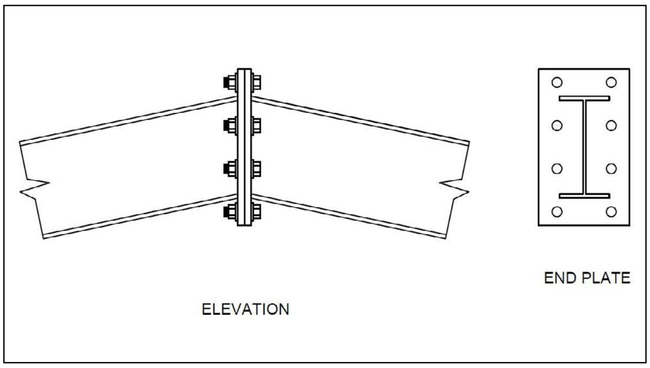

Figure 2. Model of the ridge joint considered for the study

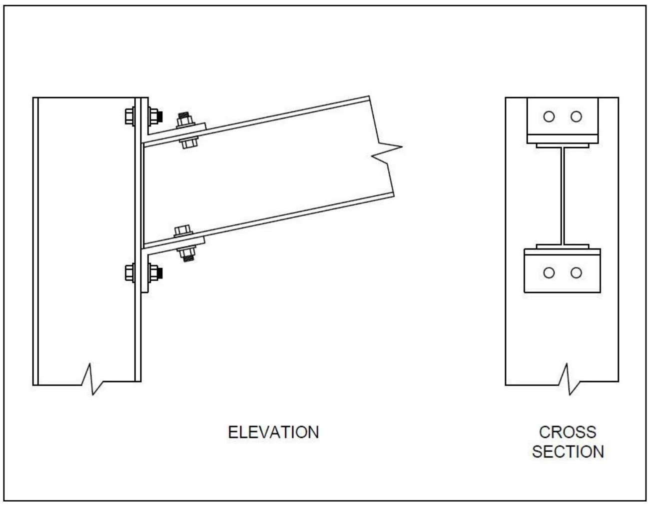

Figure 3. Model of the knee joint considered, when the rigidity value is equal to or less than $20 \cdot 10^3$ kNm/rad

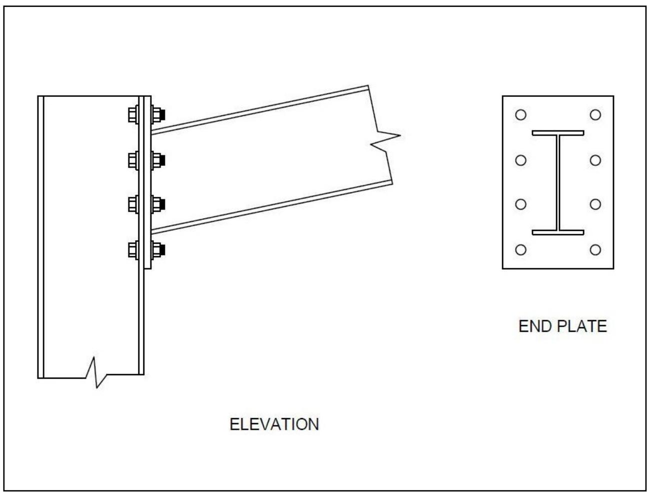

Figure 4. Model of the knee joint considered, when the rigidity value is greater than $20 \cdot 10^3$ kNm/rad

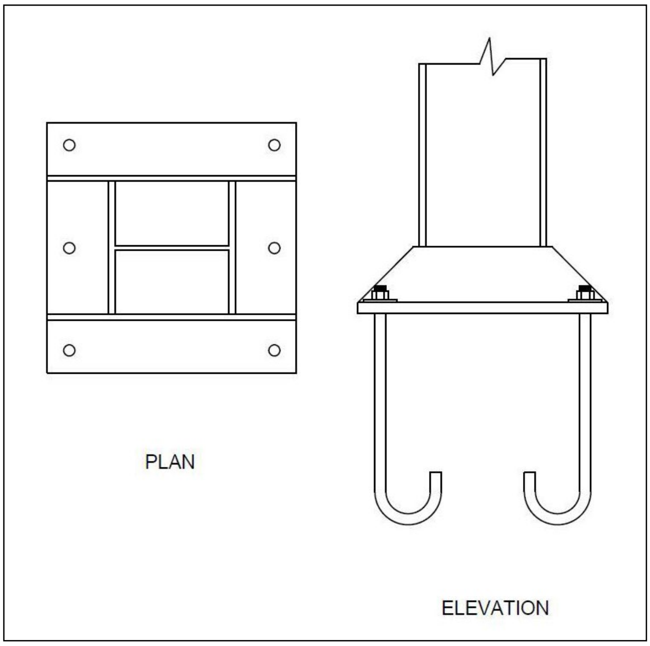

Figure 5. Model of the column base considered

Figures 2-5 show the joint types considered in this work.

The combinations and coefficients used are shown in tables 3 and 4. It should be noted that these combinations must be valid for the verification of the steel structure and also for the foundation.

Table 2. Actions considered

Kind of action	Action	Value or criterion
Permanent	Self weight	Calculated according to its profiles
Permanent	Dead weight of roof (roof material and purlins)	0.08kN/m² for folded sheet steel and 0.12 for sandwich panel. Purlins calculated according to their profiles
Permanent	Dead weight of siding material on footings	The weight of panel considered is 2.3kN/m²
Variable	Maintenance load	1 kN as concentrated load and 0.4kN/m² as uniform load (horizontal projection)
Variable	Wind load	According to the following criterion: For high load level: wind region C, roughness degree I, and internal pressure. For medium load level: B and III For low load level: A, IV and internal pressure.
Variable	Snow load (horizontal projection)	For high load level: 2.04kN/m² For medium load level: 0.7kN/m² For low load level: 0.16kN/m².

Table 3. Combinations of load cases and coefficients used for Ultimate Limit States

Combination	Permanent	Maintenance load	Snow	Ext. wind pressure	Int. wind pressure
1	1.35	0	0	0	0
2	1.35	1.5	0	0	0
3	1.35	0	1.5	0	0
4	1.35	0	0	1.5	1.5
5	1.35	0	1.5	0.9	0.9
6	1.35	0	0.75	1.5	1.5
7	1	0	0	1.5	1.5

Table 4. Combinations of load cases and coefficients used for Serviceability Limit States

Combination	Permanent	Maintenance load	Snow	Ext. wind pressure	Int. wind pressure
1	1	0 (1)	0 (1)	0 (1)	0 (1)

(1) Vertical deflection for the appearance criterion was the only limitation considered. For this reason the quasi-permanent combination was used and the value of ψ2 factors is equal to 0. For snow load it was considered that the building is at an altitude above sea level equal to or lower than 1000 m.

2.5. Costs estimation

The costs of the foundation and of the steel members were subdivided in three items: a) Steel in columns and beams (€/kg); b) Excavation of the footings (€/m³); and c) reinforced concrete footings (€/m³).

Estimation of these costs is based on reference prices obtained from nine Spanish official or widely recognized open access construction cost data bases, used for making official budgets in Spain. To homogenise the prices temporally they were updated by Ministry of Development March 2015 index means.

The three prices chosen were the average of the nine obtained from data bases for each item (Table 5).

Table 5. Costs of the foundation and of the steel members.

Budget item	Final prices used in the study
Steel in columns and beams	2.80 (€/kg)
Excavation of the footings	18.45 (€/m³)
Reinforced concrete footings	220.10 (€/m³)

Accurate estimation of the cost of the joints requires their design and sizing data for optimization and the design of the whole structure. This involves programming and implementing a complex iterative process in the software tool for calculation, followed by economical evaluation of the resultants joints. This tool has not yet been developed. To resolve this matter in the study, the cost of joints was estimated using methodologies developed by other authors:

- For column plates: Xu and Grierson (26), methodology
- For knee and ridge joints: Sanchez and Martí (9) methodology

These were taken only as a point of departure. The initial formulas and data were adapted to achieve the objectives proposed and integrate them in the design procedure.

Thus for column bases the said basic formulae have been modified using the following assumptions, and starting from Xu and Grierson (26) methodology expression [1] was obtained:

$$[1] \quad B_{2p}=C_{2c}\cdot\left(C^0+\left(C-C^0\right)\cdot R/800000\right)-C_{2c}$$

To do this, 108 column bases were designed and valued economically. More specifically, 54 were pinned and 54 were fully rigid. Coefficients C and C°, as proposed in the Xu and Grierson formula, were then estimated. Where C° is the factor which should be applied to column cost in order to obtain total cost, including the pinned plate base; and C is the similar factor to obtain the fully rigid column base (Table 6). It is also assumed that a column base with a rigidity value of 800000 kN·m/rad acts in practice as a fully rigid one. On the other hand, B_{2p} is the cost (€) of the two base plates, C_{2c} is the cost (€) of the two columns and R is the rotational joint rigidity (kN·m/rad)

Table 6. Coefficients C and C° for calculation of the cost of base plates

Portal frame			**C°**			**C**		
			Load level			**Load level**		
Span (m)	**Column height (m)**	**Roof slope (%)**	**High**	**Medium**	**Low**	**High**	**Medium**	**Low**
8	3.5	20	1.20	1.20	1.20	1.44	1.38	1.31
10	3.5	15	1.16	1.22	1.18	1.48	1.35	1.31
10	5	15	1.14	1.13	1.14	1.33	1.29	1.29
15	5	10	1.11	1.14	1.12	1.35	1.29	1.25
20	5	10	---	1.11	1.10	---	1.33	1.28
10	7	15	1.13	1.11	1.11	1.27	1.28	1.23
15	7	10	1.10	1.11	1.10	1.25	1.26	1.27
20	7	10	---	1.10	1.08	---	1.25	1.18
15	10	10	1.08	1.07	1.09	1.24	1.19	1.18
20	10	10	---	1.07	1.07	---	1.21	1.18

For knee joints the cost was obtained from formula [2], which was derived from the Sanchez and Martí (9) proposal.

$$[2] \quad C_{2k} = C_{2b} \cdot 2 \cdot (\beta^0 + \beta^I \cdot R) \cdot 0.0006308$$

Where C_{2k} is the cost (€) of two knee joints, C_{2b} the cost (€) of two beams, β° the cost coefficient (€) for pinned joints, β^I the cost coefficient (€/kN·m·rad^{-1}) for fully rigid joints and R is the initial joint rigidity value (kN·m/rad) and 0.0006308 is a dimensional coefficient (€$^{-1}$)

The parameters of Equation [2] were obtained considering the following premises:

- The variability of beam cross section and length has been taken into account. For this purpose, it must be considered that the original values of β^o_{lk} and β^I_{lk} proposed by Sanchez and Martí were calculated for a beam with a length of 7.3 m and an IPE-450 profile. For other beam profiles and lengths these values have been increased or decreased proportionally.
- The values of β^o_{lk} and β^I_{lk} (from the Sanchez and Martí proposal) have been updatedby means of the official index of construction costs of INE (National Statistics Institute of Spain). The updated is made from 2004 which is the reference proposed by Sanchez and Martí to 2013 because this is the last year which was recorded by the INE. These updated values have been termed β° and β^I, respectively.
- Two types of joint were selected, depending on their rigidity. When the rigidity value is higher than $20{\cdot}10^3$ kN·m/rad the joint type considered was the extended end plate (Figure 4), and when the rigidity value is equal to or lower than $20{\cdot}10^3$ kN·m/rad (Figure 3), flange cleat joints were considered .

Table 7 shows the updated β° and β^I values used in the study.

Table 7. Updated β° and β^I values used in the study

Kind of joint	Updated β° (€)	Updated β^I (€/ kN·m·rad-1)
Flange cleats joint	40.897	0.001000
Extended end plate joint	61.702	0.000429

A similar methodology to that for knee joints was used for ridge joints. The specific assumptions used for these joints were: a 12% decrease of cost was estimated with respect to knee joints; only the extended end plate type was considered; and ridge joint cost was estimated including plates, welds and bolts. Equation [3] was obtained.

$$[3] \quad C_{wr} = C_{2b} \cdot 2 \cdot (\beta^0 + \beta^I \cdot R) \cdot 0.0005538$$

Where C_{wr} is the cost of the whole ridge joint (€) and the other parameters are the same as in formula [2]. The values of β° and β^I are also shown in table 7 for extended end plate joints.

2.6. Method of automatic calculus used and number of cases analyzed

Specific software was developed by the authors for the automatic calculation of the whole process, named "CalculoRigideces V 1.04". To run structural design, this application uses the same calculation engine as the commercial software Metalpla XE_5 (27); and to determine buckling lengths of the members under each assumption this application utilizes the same calculation engine as commercial

"Metalbuckling" software. Metalpla XE_5 (27) applies matrix methods to structural design.

"CalculoRigideces V 1.04" is fed from an Excel file which contains the records, one for each structural case as well as the fields necessary to define the portal frame and its circumstances. This file, in turn, receives the results of the structural calculus. Results data are then processed to finding the cases which fulfil final structural optimization requirements. Each case is defined by its combination of rigidity values. Only one case is the best for each different portal frame geometry and load level. This case is the most economical one.

The process of structural design in an optimization context takes place in the following order, figure 6:

1. Data input.
2. Process initiation.
3. Assembling the geometric matrix of structure.
4. Automatic buckling coefficient calculus.
5. Member ultimate limit states calculation loop.
6. Member serviceability limit states calculation loop.
7. Cycle for optimum structure selection for all load combinations.
8. Base plate design.
9. Foundation design.
10. Recording results of structure.
11. Optimization process including joints.
12. Final results. The optimum rigidity combination is found.

46656 cases were analysed. This number is obtained by means of the product of 10 geometric portal frame cases, each at 3 load levels and the 1728 different rigidity combinations of joints. It is necessary to stress that for cases of a 20 m portal frame span the high load level has been omitted. The product of geometric portal frames and load level is therefore 27 instead of 30.

2.7. Consideration of instability and buckling.

The analytical method used in this study is elastic global analysis in first order theory. The structure was considered to be translational in the portal frame plane and non-translational in the plane perpendicular to the portal frame, due to the existence of a cross-shaped bracing system. For these reasons and because the frame is a basic structure with only one storey, the verification of its stability in both planes was carried out by means of individual stability checks of equivalent members using appropriate buckling lengths. The critical factor method of global buckling α_{crit} was applied to calculate buckling lengths. This methodology was included in the software for automatic calculation, thereby integrating it in the overall calculus. The value of α_{crit} is obtained by matrix methods. (Argüelles Álvarez *et al.* (28); Argüelles Álvarez *et al.* (29))

2.8. Consideration of the behaviour model of joints

The behaviour model of joints used in this study is linear, equivalent to a bi-linear without reaching the design moment of joint (maximum or yield moment)

3. RESULTS AND DISCUSSION

All results were collected, summarized and graphically represented for analysis. It must be emphasized that this involved creating a collection of graphics which represent the final cost of each portal frame depending on its combination of joint rigidity values. Cost is the output variable, and the three input variables were column base rigidity, knee joint rigidity and ridge joint rigidity. In total there were 4 variables, which makes it hard to show them graphically. To resolve this 12 graphics were drawn for each portal frame type and load level, each distinguished by the rigidity value of its ridge joints. Each one of them represents the values of the rigidity of knee joints in one axis and column base rigidity values in a perpendicular axis. The cost associated with these 3 values of rigidity was represented on a gray scale. Each grade represents a cost range. Figures 7, 8 and 9 show three 3D view examples which are characteristic of three models of cost distribution.

Thanks to their synthetic nature this series of graphics has been a crucial tool in analyzing the results and reaching conclusions. Interpretation of the information contained in the graphics may be summarized as follows:

- A pinned ridge joint is never favourable (lower cost).
- The lower cost zones generally correspond to low column base, knee joint and ridge joint rigidity values. This is due to the favourable stress distribution achieved and the lower cost of joints.
- The rigidity of knee joints generally has more impact on the final cost than column base rigidity. For high load levels the impact of these two kinds of joint may be considered similar.

Table 8. Average of the contribution of each structural component studied to the total cost

Structural component studied	Average contribution to the total cost (%)
Columns	27
Beams	38
Joints	17
Column bases	4
Knee joints	7
Ridge joints	6
Footings	17
Excavation of the footings	1

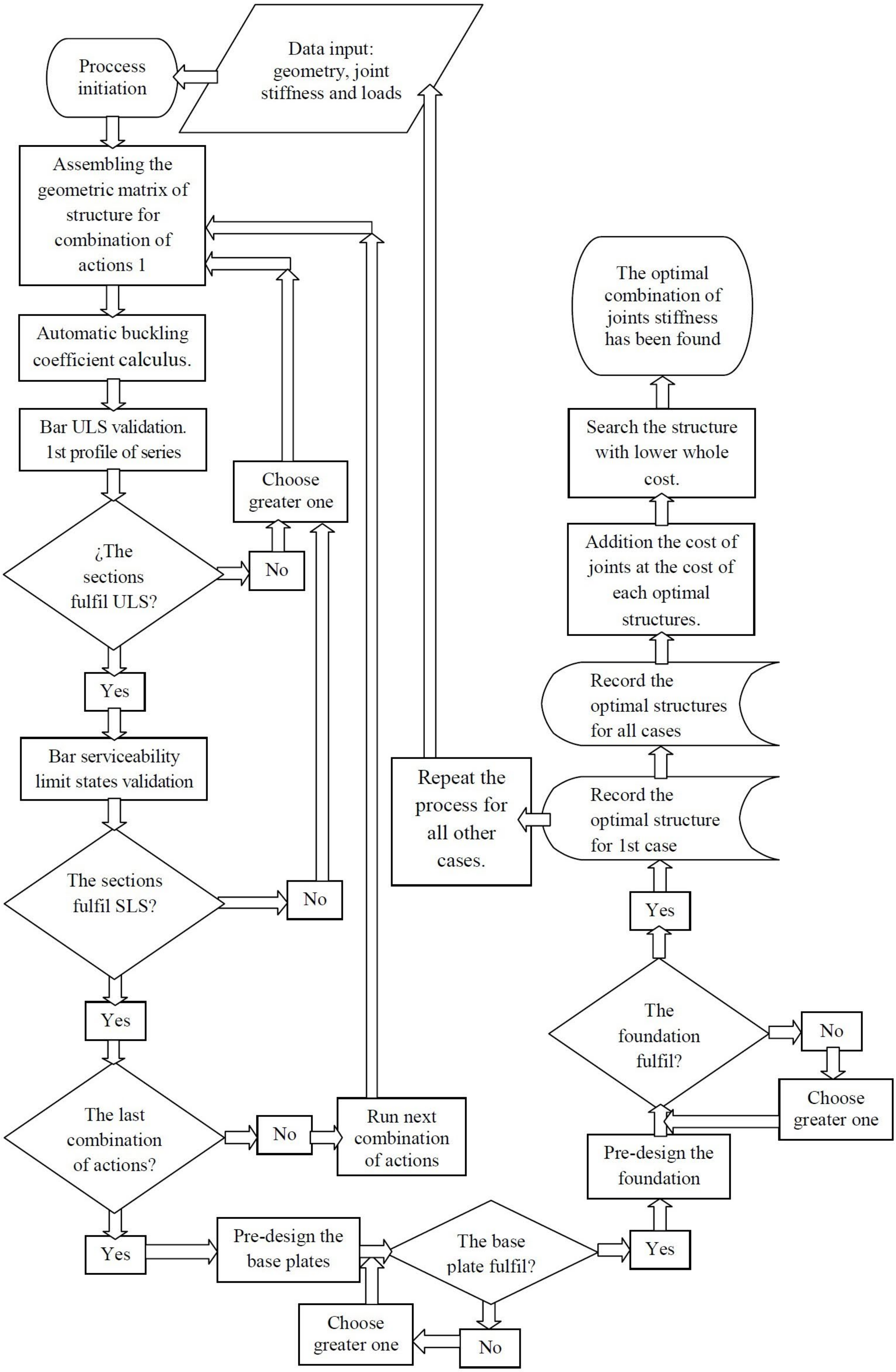

Figure 6. Flow chart of the process of structural design in an optimization context.

Table 9. Joint rigidity combination corresponding to the lower cost portal frames and results of the comparison between the cost of portal frames with semi-rigid joints and optimum rigidity combinations and two kinds of traditional type portal frames.

Span (m)	Column height (m)	Roof slope (%)	Load level	Rigidity joints combination (1) (x10³ kNm/rad)	Economic advantage over fully rigid portal frame (%)	Economic advantage over pinned column bases portal frame (%)
8	3.5	20	High	0 / 5 / 10	25	43
			Medium	5 / 5 / 5	25	27
			Low	5 / 5 / 5	35	35
10	3.5	15	High	5 / 5 / 10	40	16
			Medium	40 / 10 / 5	22	24
			Low	5 / 5 / 5	31	26
10	5	15	High	0/20/10	27	19
			Medium	0/5/5	22	28
			Low	5/5/15	25	23
15	5	10	High	0/80/10	25	11
			Medium	0/20/5	13	14
			Low	5/5/40	26	23
20	5	10	Medium	0/40/20	18	7
			Low	5/5/200	27	14
10	7	15	High	0/10/10	27	14
			Medium	5/20/5	23	27
			Low	5/10/5	26	22
15	7	10	High	0/15/15	20	13
			Medium	5/10/15	18	16
			Low	10/10/5	27	18
20	7	10	Medium	5/40/5	10	10
			Low	10/10/100	22	12
15	10	10	High	5/10/20	9	11
			Medium	5/20/10	10	14
			Low	5/10/5	18	21
20	10	10	Medium	5 / 10 / 80	22	22
			Low	5 / 10 / 15	21	15

(1) The first number corresponds to column base rigidity, the second to knee joint rigidity and the third to ridge joint rigidity. 0 / 5 / 10 as an example, *means the following combination of rigidities: pinned in column base, 5·10³ kN·m/rad in knee joint and 10·10³ kN·*/rad in ridge joint.

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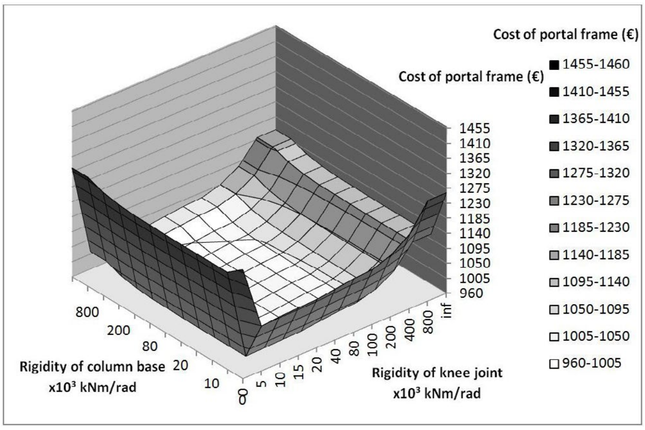

Figure 7. 3D view of the graphic corresponding to 8 m span, 3.5 m column height and 20% slope portal frame at low load level. Rigidity of ridge joint 80·10³ kNm/rad

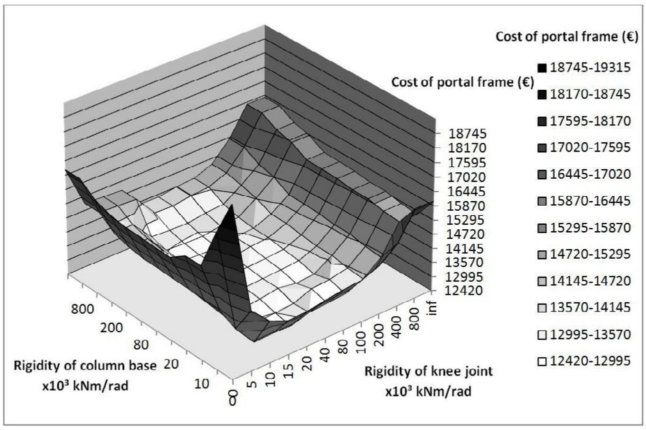

Figure 8. 3D view of the graphic corresponding to 15 m span, 10 m column height and 10% slope portal frame at high load level. Rigidity of ridge joint 200·10³ kNm/rad

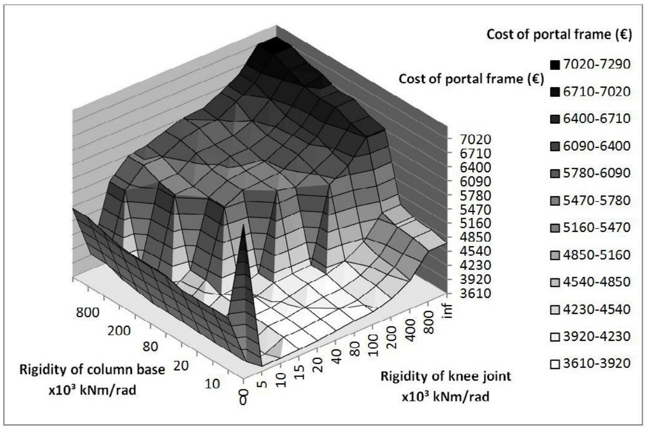

Figure 9. 3D view of the graphic corresponding to 10 m span, 3.5 m column height and 15% slope portal frame at high load level. Rigidity of ridge joint 10·10³ kNm/rad

On the other hand, a comparative study of semi-rigid portal frames and the traditional types was undertaken. The two traditional types were: a) Portal frame with all joints fully rigid; b) Portal frame with pinned column bases and the other joints fully rigid. Table 9 summarizes the lower cost combinations of joint stiffness for each portal frame and load level. It also shows the results of the comparative study with traditional types.

Table 8 also shows the average contribution to the total cost of each structural component studied. This average was calculated from all of the cases studied.

One issue for discussion has to be underlined. This consists of the real possibility of making joints which have the desired rigidity. Moreover, the desired rigidity could be limited by the joint strength (Eröz *et al.* (30)). According to Eurocode 3, the rotational stiffness of a joint may be determined from the following factors: the joint geometry and its components, the lever arm, the level of the design moment resistance of the joint and partial stiffness of its basics components. The optimum rigidity values shown in Table 9 are theoretical. This means that these values may or may not be achieved by the actual joints. In some cases it is not physically possible to make a joint with a specific rigidity, or it is not possible fulfil the requirement of strength and stiffness at same time.

It can therefore be said that the theoretical optimum values of rigidity may not be possible in practice. This should be taken into account in structural design. Nevertheless the useful is found, because the stiffness of realistic joints are also included in the research range and hence in its helpful results. For the use of the results by engineers they might study the optimum stiffness combinations zones of the graphics according to the stiffness and strength that their joints setting allow arise.

4. CONCLUSIONS

The cost of joints is a major part of the whole cost of the kind of structures studied; more specifically they amount to an average of around 17% of the total cost; but they are not only important due to this. Their variability and affect on the behaviour of the whole structure makes them essential elements for cost optimization.

It is clear that the choice of one rigidity value or another for each joint of the portal frame means they will respond differently. This fact constitutes in itself a source of variation. This variability implies that for one structure there are many solutions and that one of these is the best. This too is an optimization source.

A combination of joint rigidity values was obtained for each portal frame geometry and its associated load level for the lowest cost. In general, it can be said that lower cost structural solutions have low joint rigidity values, understanding these as 5·10³ or 10·10³kNm/rad.

Portal frame load level also influences the relevance of joint rigidity on the cost. Thus a low load level underscores the influence of high rigidity knee joints in increasing cost; at medium load level the high or low rigidity of knee joints influences the increased cost, as do column bases but to a lesser degree; and for a high load level, high rigidity knee joints and column bases influence the increase in cost equally.

To quantify the potential benefits of applying a combination of portal frame joint rigidity values, it is necessary to compare the results of these with those of traditional type portal frames.

Theoretically and according to the cases studied, average cost reductions of around 18% may be achieved. In the best case the reductions may be 35% and in the worst case 7%. It also has to be underlined that benefits are higher when portal frame span is smaller and their load level is lower, too. In other words, the benefits of applying the optimum combinations of joint rigidity are greater for small industrial buildings under low load.

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