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PARAMETRIC STUDY OF LATTICE TOWERS ON THE INFLUENCE OF WIND ACTION FOR DIFFERENT TYPOLOGIES OF BRACING

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Abstract

The telecommunications sector is a fundamental pillar of today's society. The answer to the need for constant communication is based on a network of structures, that allows the uninterrupted functioning of the communication system. Among these structures of the communication infrastructure are the so-called telecommunications towers, which are fundamental elements for the functioning of the system. Therefore, given the importance and actual demand for these structures, there is a great need for a deep knowledge of their behaviour, to find more economically viable and safer solutions. Since the wind action is in fact the most significant and conditioning for the design of this type of structures, this work focusses on the study of different bracing schemes, which were divided into three groups differing by specific parameters or even locations. Starting from a base or initial model of a lattice tower designed according to Eurocodes (EC1 and EC3), five conceptual tower models were redesigned with different lay-out schemes, but without changing the territorial location and their height and width. These models were analysed and compared to understand how different bracing schemes can affected the wind action determination and what are their consequences on the behaviour of the structures and their design.

Keywords: Lattice towers, wind loading action, EN 1991-1-4, EN 1993-3-1, parametric study on bracing.

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1 INTRODUCTION

The development of today's society is also strongly linked to the ease and speed of communication and information sharing, and so it is not difficult to understand the importance of telecommunications towers, commonly present in the urban landscape and elsewhere. These structures, normally of great height, have as main objective the placement of transmission equipment on its top that allows the adequate transmission of signals. With an increasing use of mobile phones and the offer of services such as channel television, there is a greater need for the implantation of these structures to guarantee a good quality of all the services provided [1]. But despite the great importance of telecommunications towers to society, these structures exhibit statistically a higher number of failures when compared with many other structures [2]. Thus, it is essential to understand the behaviour of these structures, so that it is possible to design them as economically viable as possible, but still complying with safety criteria.

This work focusses on the study of different bracing schemes, which can be divided into three groups differing by specific parameters or even locations; a change or modification done to a specific tower parameter causes re-sizing of structural members to satisfy the structural design conditions for the tower wind actions at the implantation site [3]. Starting from a base model of a lattice tower designed according to Eurocodes (EC1 and EC3), conceptual tower models were redesigned with different lay-out of bracing schemes, but without changing the territorial location and the tower height and width. These models were analysed and compared to understand how different bracing schemes can affected the wind action determination and what are their consequences on the behaviour of the structures and their design.

2 DESCRIPTION OF CONCEPTUAL TOWER MODELS

2.1 Initial Tower Model for Reference Comparison

The initial model of the tower, predesigned by the industrial partner Metalogalva - Irmãos Silva SA according to this metalo-mechanical company experience on lattice towers and its industrial production characteristics (including delivery and assemblage at specific locations), consists of a self-supporting lattice tower with a triangular section 100 meters high and 15 meters wide at the base.

The tower can be divided into two distinct tower segments: the first or lower segment, from the base to about two thirds of the height, up to 72.5 meters, uses the K-typology scheme as the main bracing system in elevation along the height; the second or upper segment, in the last 27.5 meters, uses lattice diagonal bracing because this tower segment width is much smaller.

The diaphragms, which constitute horizontal stiffening zones or horizontal bracing systems, are spaced 6 meters apart up to 72.5 meters in height of the first tower segment; in the last 27.5 meters, the diaphragms of the second tower segment are spaced 3 meters apart.

Table 1 indicates the characteristics of the initial tower model; and Figure 1 shows the general 3D layout.

Table 1 : Characteristics of the initial tower model

Total height	100 m
Top width	3 m
Base width	15 m
Number of sub-segments (12+10)	22
Type of profile used	Circular
Steel class	S 355
Ultimate stress (f_u)	510 MPa
Yield stress (f_y)	355 MPa
Elasticity modulus (E)	210 000 MPa
Distortion modulus (G)	81 000 MPa
Poisson coefficient (v)	0.3
Coefficient of thermal expansion (α)	12 x 10^{-6} / ºC

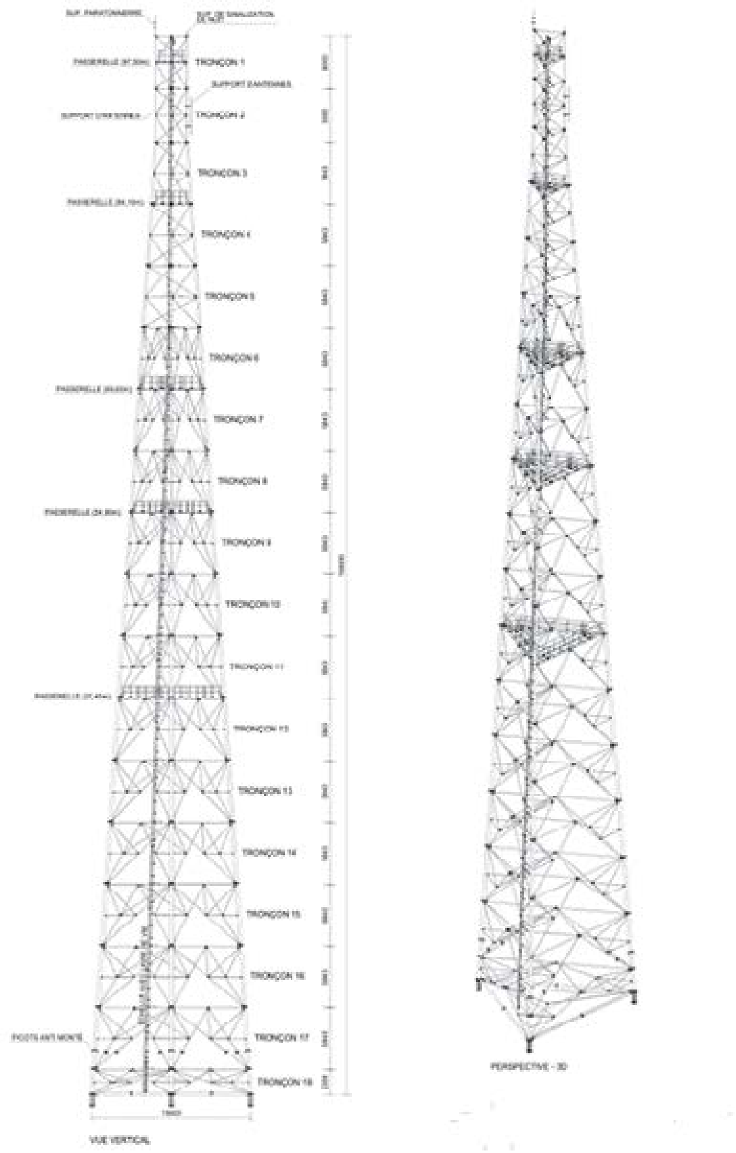


Figure 1 : Initial tower model.

2.2 Tower Model A

This tower model A (shown in Figure 2) is very similar to the initial tower model, the only change occurring in the upper tower segment with a truss system where a new circular profile member was added in the opposite direction to the existing one; therefore, making it a cross (or X) bracing system instead of the diagonal bracing of the initial tower model.

This alteration attributes a greater stiffness to the altered upper tower segment, increasing its torsional stiffness; however, with the addition of the diagonals, the exposure area to wind actions increases.

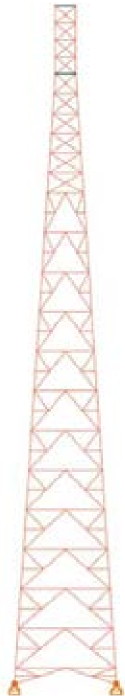


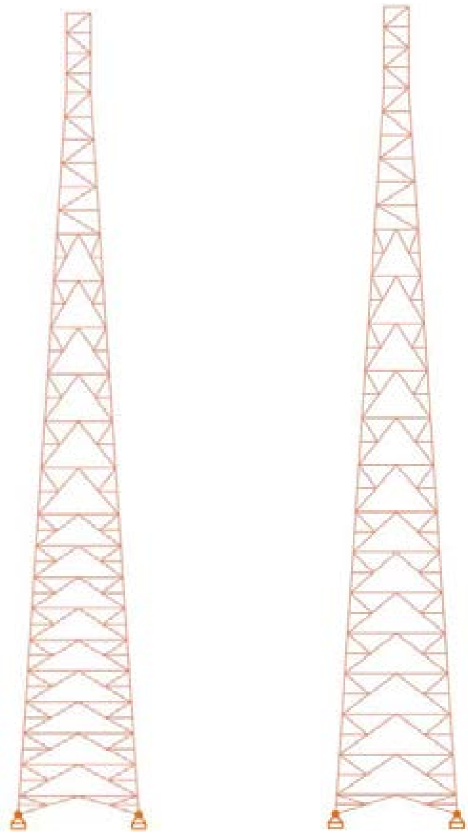
Figure 2 : Tower model A.

2.3 Tower Model B

In tower model B (shown in Figure 3 (a)), the change made consists of varying the spacing between the horizontal diaphragms bracing system. Such change occurred only in the first third part of the structure, up to 37 meters in height, and the spacing chosen between diaphragm levels is 4 meters.

2.4 Tower Model C

The tower model C (shown in Figure 3 (b)) is like tower model B, but now studying the variation of results for a chosen spacing of 5 meters between horizontal diaphragms in the first 37 meters of the tower structure; so, initial tower model has horizontal diaphragm bracing system with 6 meters spacing, while for tower models B and C, such spacing is of 4 meters and 5 meters respectively.



(a) (b)

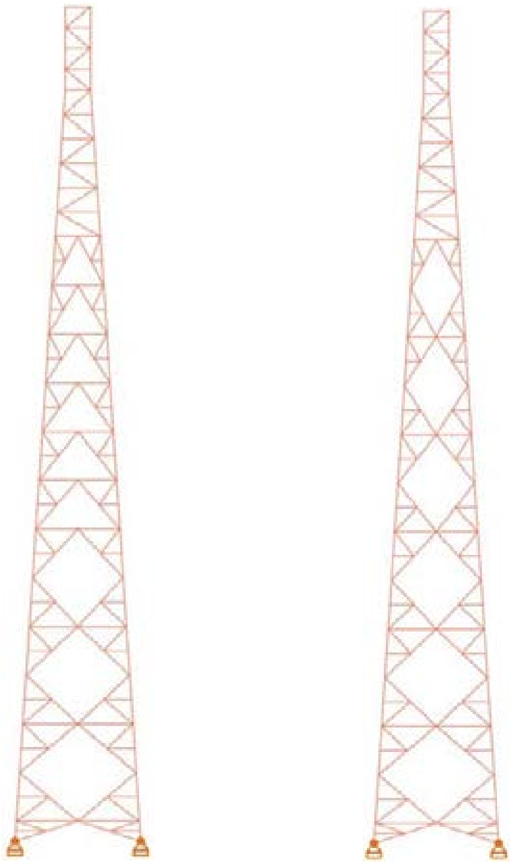
Figure 3 : (a) Tower model B ; (b) Tower model C.

2.5 Tower Model D

The development of the tower model D aims to analyse the structure's behaviour in case the main bracing system in elevation is altered in the first third part of the structure, up to 37 meters in height, from the original “K” bracing system to a discontinuous bracing system; as shown in Figure 4 (a), this tower model D still presents the continuous horizontal diaphragm bracing system at specific horizontal intersections.

2.6 Tower Model E

The tower model E shown in Figure 4 (b), like the tower model D, also aims to study how some alterations of the bracing system influences the operation of the tower, using another variation of discontinuous main bracing system in elevation. In this model tower E, the main “K” bracing system that exists in elevation in the original tower was replaced entirety up to 72.5 meters in height, by a discontinuous bracing locking system in elevation, but still with continuous horizontal diaphragm bracing system at specific horizontal intersections.



(a) (b)

Figure 4 : (a) Tower model D ; (b) Tower model E.

In Table 2 a brief synthesis is given about the six tower models designed and analysed in this parametric study, but still complying with the metalo-mechanical company experience on lattice towers and its industrial production characteristics. For each design alternative, emphasis is given to the final total weight of the tower (directly related to costs) and to the exposed area to wind in each tower face or incidence plane to wind.

Table 2 : Final weight and exposed areas, for alternative tower models

Tower label	Total weight of tower (kgf) ; 1 kgf ≈ 0.01 kN	Exposed area per face (m^2)
Initial tower model	22354.13	95
Tower model A	23366.31	95.5
Tower model B	29610.04	114
Tower model C	27087.03	108
Tower model D	21928.05	93
Tower model E	21995.66	90

3 WIND ACTIONS ON THE TOWERS

The design guidelines for the regulatory normative of the wind actions on the towers, were the corresponding parts of Eurocodes EC1 and EC3 namely EN 1991-1-4 [4] and 1993-3-1 [5]. Additional comprehensive bibliography used as support were the book chapter of Barros *et al.* [6] and the article of Calostescu [7]. Also the graduation master's thesis of the second co-author [8] details the remaining expressions and calculations.

3.1 Wind velocity and dynamic wind pressure

The tower is located in Zone B mentioned in NP EN1991-1-4/NA [4], to which corresponds a reference wind base velocity of 30 m/s; the adopted orography coefficient was $c_o(z)$=1.

For ground category II, parameters z_0 and z_{min} are respectively z_0=0.05 m and z_{min}= 2 m.

The turbulence coefficient K_I used, was the recommended value of 1. Using specific mass of air ρ_{air} = 1.25 kg/m^3 – mentioned in clause 4.5 (1) of Eurocode 1 - EN 1991-1-4 [4] – and resorting to the expressions of EC1 [4] and EC3 [5] with respect to characterization of wind actions (by one of the two methodologies in Eurocode annexes), or to the expressions also used in the abovementioned comprehensive bibliography of Barros *et al.* [6] and Calostescu [7], Table 3 can be systematically obtained. The last two columns in this table (calculation procedure of wind actions - step I) present respectively the dynamic wind pressure (in Pa) and the wind peak velocity (in m/s) at 18 section levels (limiting distinct tower panels), in which the towers were discretized along height at specific elevations z (in m) for improving more accurate determination of distributed wind related quantities.

Table 3 : Calculation procedure - step I

Section	z (m)	v_m (m/s)	c_r (z)	I_v	q_p (Pa)	V_{peak} (m/s)
1	2,37	21,994	0,733	0,259	850,818	36,895
2	8,21	29,076	0,969	0,196	1253,478	44,783
3	14,06	32,143	1,071	0,177	1447,278	48,121
4	19,9	34,123	1,137	0,167	1578,664	50,258
5	25,756	35,593	1,186	0,1601	1679,394	51,837
6	31,6	36,758	1,225	0,155	1761,169	53,084
7	37,44	37,725	1,258	0,15109	1830,273	54,115
8	43,29	38,553	1,285	0,148	1890,361	54,996
9	49,14	39,275	1,309	0,145129	1943,522	55,764
10	55	39,917	1,331	0,143	1991,32	56,446
11	60,83	40,491	1,349	0,141	2034,501	57,054
12	66,68	41,015	1,36	0,139	2074,216	57,609
13	72,53	41,494	1,383	0,137	2110,888	58,116
14	78,37	41,936	1,398	0,136	2144,913	58,582
15	84,22	42,346	1,411	0,1346	2176,762	59,015
16	90,07	42,729	1,424	0,133	2206,661	59,419
17	95,06	43,036	1,434	0,132	2230,8	59,743
18	100	43,325	1,444	0,131	2253,588	60,047

3.2 Wind force coefficient

The coefficient of wind force of a lattice metallic tower c_f, depends on the index of filled parts φ defined as the ratio between the solid area per panel and the area of the panel if it was completely filled. In this determination of the coefficient of wind force required for the calculation of wind actions, it is conservatively assumed that the tower is subjected to a super-critic wind flow. For lattice towers of triangular cross section, made of equilateral triangles, the wind causes the maximum drag when its direction is normal to the face, as [9]. So it is going to be considered in these analyses a null or zero angle of attack, that is the wind acts normal to the face of the tower (θ = 0º) and then the wind incidence factor K_θ=1.

Table 4 details for the section levels associated with tower panels, the calculation procedure of wind actions - step II: index of filled parts, and corresponding wind force coefficient.

Table 4 : Calculation procedure - step II

Section	z (m)	φ	c_f
1	2,37	0,156	1,083
2	8,21	0,091	1,088
3	14,06	0,094	1,088
4	19,9	0,096	1,088
5	25,756	0,099	1,087
6	31,6	0,1	1,087
7	37,44	0,105	1,087
8	43,29	0,103	1,087
9	49,14	0,109	1,08
10	55	0,110	1,086
11	60,83	0,117	1,086
12	66,68	0,128	1,085
13	72,53	0,133	1,085
14	78,37	0,144	1,084
15	84,22	0,143	1,084
16	90,07	0,165	1,084
17	95,06	0,153	1,083
18	100	0,153	1,083

3.3 Structural coefficient

For the determination of the structural coefficient $c_s c_d$ the dynamic parameters of the tower were obtained using a finite element method model. The first vibration mode has a frequency very close to unit, so that (in Eurocodes symbology) n1=1; also, for the logarithmic decrement of structural damping, the value δ_s = 0.05 was assumed. The remaining intermediate values necessary for the calculation of the structural coefficients are shown in the following Table 5 through Table 7; for more details, refer to [6] [7] [8].

Table 5 : Calculation procedure - step III (a)

L (z_s)	m_e(Kg/m)	δ_s
209,181	669,5	0,05

Table 6 : Calculation procedure - step III (b)

Section	R_h	R_b	n_h	n_b	S_l	f_l	δ	K_p
1	0,037	0,254	26,771	3,933	0,026	12,174	0,305	3,082
2	0,049	0,355	20,250	2,817	0,032	9,209	0,371	3,176
3	0,055	0,416	18,318	2,405	0,034	8,330	0,385	3,220
4	0,058	0,469	17,255	2,131	0,035	7,847	0,384	3,254
5	0,060	0,522	16,543	1,914	0,036	7,523	0,377	3,283
6	0,062	0,579	16,018	1,728	0,037	7,284	0,364	3,311
7	0,064	0,640	15,608	1,562	0,038	7,097	0,349	3,338
8	0,065	0,709	15,273	1,410	0,038	6,945	0,332	3,365
9	0,067	0,789	14,992	1,267	0,038	6,817	0,313	3,393
10	0,068	0,884	14,750	1,131	0,039	6,708	0,293	3,421
11	0,069	0,997	14,541	1,003	0,039	6,612	0,271	3,451
12	0,070	1,138	14,356	0,879	0,040	6,528	0,249	3,483
13	0,070	1,320	14,190	0,758	0,040	6,453	0,225	3,517
14	0,071	1,562	14,040	0,640	0,040	6,385	0,201	3,553
15	0,072	1,903	13,904	0,526	0,040	6,323	0,177	3,593
16	0,073	2,419	13,780	0,413	0,041	6,266	0,151	3,636
17	0,073	2,436	13,681	0,410	0,041	6,222	0,152	3,637
18	0,074	2,453	13,590	0,408	0,041	6,180	0,153	3,639

Table 7 : Calculation procedure - step III (c)

Section	z (m)	B (m)	B^2	R^2	$C_S C_d$
1	2,37	14,69	0,618	0,004	0,706
2	8,21	13,91	0,618	0,007	0,746
3	14,06	13,13	0,618	0,010	0,761
4	19,9	12,35	0,618	0,012	0,771
5	25,756	11,57	0,618	0,015	0,778
6	31,6	10,79	0,618	0,018	0,784
7	37,44	10,01	0,618	0,022	0,789
8	43,29	9,23	0,618	0,026	0,794
9	49,14	8,45	0,618	0,032	0,798
10	55	7,67	0,618	0,039	0,803
11	60,83	6,9	0,618	0,049	0,807
12	66,68	6,12	0,618	0,062	0,812
13	72,53	5,34	0,618	0,081	0,817
14	78,37	4,56	0,618	0,110	0,823
15	84,22	3,78	0,618	0,155	0,832
16	90,07	3	0,618	0,233	0,849
17	95,06	3	0,618	0,236	0,850
18	100	3	0,618	0,239	0,851

3.4 Wind forces

The wind forces are calculated using the following equation

$$F_W = c_s c_d \cdot c_f \cdot q_p(z_e) \cdot A_{ref} \tag{1}$$

which was shown in detail and with its used explained, in the third chapter of [8].

In Table 8 are presented the quantities used for the calculation of wind force, the wind force value, and the wind force value per node in each panel; again, for more details, refer to [6] [7] [8].

Table 8 : Calculation procedure - step IV

Section	z (m)	$c_s c_D$	I_v (z)	q_p (Pa)	A_{ref} (m^2)	F_W (kN)	F_w per node (kN)
1	2,37	0,706	0,259	850,818	5,466	3,56	1,19
2	8,21	0,746	0,196	1253,478	7,646	7,78	2,59
3	14,06	0,761	0,177	1447,278	7,452	8,93	2,98
4	19,90	0,771	0,167	1578,664	7,116	9,42	3,14
5	25,76	0,778	0,160	1679,394	6,938	9,86	3,29
6	31,60	0,784	0,155	1761,169	6,623	9,94	3,31
7	37,44	0,789	0,151	1830,273	6,401	10,05	3,35
8	43,29	0,794	0,148	1890,361	5,787	9,44	3,15
9	49,14	0,798	0,145	1943,522	5,644	9,52	3,17
10	55,00	0,803	0,143	1991,320	5,212	9,05	3,02
11	60,83	0,807	0,141	2034,501	4,968	8,86	2,95
12	66,68	0,812	0,139	2074,216	4,851	8,86	2,95
13	72,53	0,817	0,137	2110,888	4,445	8,31	2,77
14	78,37	0,823	0,136	2144,913	4,169	7,98	2,66
15	84,22	0,832	0,135	2176,762	3,484	6,84	2,28
16	90,07	0,849	0,133	2206,661	3,266	6,62	2,21
17	95,06	0,850	0,132	2230,800	2,298	4,72	1,57
18	100,00	0,851	0,132	2253,588	2,298	4,77	1,59

3.5 Wind forces on equipment auxiliary structures

For the calculation of the forces acting on equipment auxiliary structures, an equipment exposed area of 30 m^2 was allowed in the last 10 meters from the top of the tower. Account was also taken on the existence of an access staircase throughout the height of the tower, as well as 0.30 m of cablings width across the height.

Table 9 shows the values of wind forces in auxiliary structures, and their section level of actuation.

Table 9 : Wind forces on equipment auxiliary structures

Section	F_W (kN)	F_w per node (kN)
7	1,35	0,45
9	1,20	0,40
16	11,8	3,83
17	11,64	3,88
18	11,72	3,91

3.6 Combinations of actions

The combinations of actions used in this parametric study for the ultimate limit state (ULS) are shown in Tables 10-11-12. Tables 10 and 11, detail the conditioning combinations for the determination of generalized forces; and Table 12 details the conditioning combination for the determination of displacements.

Table 13 details the conditioning combination of actions used in this parametric study for the determination of displacements under the serviceability limit state (SLS).

For a better understanding of the combinations detailed, the nomenclature used and its meaning is the following: PP refers to self-weight; Wind_Y+ refers to wind acting along the direction Y+ ; Wind Y- refers to wind acting along the direction Y- ; Wind_X refers to wind acting along the direction X ; PLAT SW refers to platform self-weight; PLAT LL refers to platform live load.

Table 10 : Combination of actions for ULS – generalized forces (1st combination)

Factor	Action
1.2	PP
1.2	PLAT SW
1.12	PLAT LL
1.6	Wind_Y+

Table 11 : Combination of actions for ULS – generalized forces (2nd combination)

Factor	Action
1.2	PP
1.2	PLAT SW
1.12	PLAT LL
1.6	Wind_X

Table 12 : Combination of actions for ULS – displacements

Factor	Action
1.2	PP
1.2	PLAT SW
1.12	PLAT LL
1.6	Wind_Y-

Table 13 : Combination of actions for SLS – displacements

Factor	Action
1.0	PP
1.0	PLAT SW
0.64	Wind_Y-

4 ANALYSIS OF RESULTS

Table 14 shows the values of the generalized reactions at the base of the towers, obtained at each tower model analysed in accordance with the ULS combinations of Tables 10 and 11. These generalized reactions are respectively the force reactions in the horizontal base plane (F_x and F_y) and the vertical force reaction F_z; as well as the two moment reactions M_x and M_y at the base of the towers.

Table 14 : Values of Reactions at the base of the towers for ULS

Tower label	Fx (kN)		Fy (kN)		Fz (kN)		Mx (kNm)	My (kNm)
	Max	Mín	Max	Mín	Max	Mín		
Initial tower model	72	-134	142	-170	1330	-1148	17294	15855
Tower model A	74	-139	148	-176	1401	-1212	18213	16774
Tower model B	77	-142	142	-178	1378	-1144	17914	16041
Tower model C	77	-140	149	-181	1381	-1164	17953	16178
Tower model D	74	-134	144	-171	1321	-1141	17173	15817
Tower model E	71	-131	140	-167	1321	-1141	17173	15772

From the analysis of the results of this parametric study, each tower identity or parameter of change can be easily distinguished. Tower model A has the highest bending moments (M_x and M_y) at the base due to the increase in the wind force because of the increase in the exposure area of this model has been made at the top of the structure, greatly influencing the value of the moments at the base.

In the tower models B and C, the two bending moments at the base are superior to the moments at the base of the initial tower model: for reasons already explained above, and also because the compromise between increased wind action and increased stiffness (due to the resizing) did not prove to be beneficial at this point.

In the tower models D and E, the two bending moments at the base are slightly lower than the moments at the base of the initial tower model: because the action of the wind on these tower models is slightly less than the action of the wind on the original tower, thus generating smaller moments.

Regarding the values of the generalized displacements (lateral displacements and torsional rotation through vertical global axis) in accordance with the SLS combination of Table 13, their results are presented in Table 15 for the top as well as for sections at two-thirds and one-third of the tower height (H).

Table 15 : Values of lateral displacements and rotations, for the SLS

Tower model	Lateral displacement (cm)			Rotation along vertical axis (º)		
	Top	2/3 H	1/3 H	Top	2/3 H	1/3 H
Initial tower	31,1	14,7	3,9	0,526	0,321	0,134
Tower A	32,9	15,6	4,1	0,497	0,351	0,139
Tower B	32	15,2	4,1	0,532	0,327	0,123
Tower C	32	15,1	4	0,531	0,330	0,128
Tower D	31,2	14,8	4	0,523	0,321	0,131
Tower E	32,1	15,5	4	0,529	0,325	0,160

In general, it is observed that the difference in lateral displacements is not significant: of the order of 1 cm in the top section for models A-B-C-E, and 1 mm for model D; much smaller differences at the other tower sections. With regards to rotation along vertical axis, all the tower models have rotation values quite identical, except the top rotation of tower model A; in fact, for this tower model A the maximum rotation is 5% to 7% lower than maximum rotation of the other tower models. This could be expected because the increase of the stiffness of the top third part of the tower structure (above 72.5 m) by adding an additional diagonal member, converting the initial diagonal bracing into a cross X-bracing, leads to an expected reduction of torsional rotations at the top tower levels.

5 CONCLUSIONS

All the proposed tower models were resized or redesigned, leading to changes in the type of circular profile used in various sections and in the total number of elements used. These changes, besides leading to different structural responses when the towers were subject to wind action (as previously explained and analysed), also lead to different costs of the models proposed tower structures. Table 16 shows an estimate of the total weight of each tower and an estimate of the price of each tower, considering a market value for the steel price of around 2.5 euros per kilogram-force of weight.

In view of the foregoing considerations -- both in terms of the generalized reaction values and generalized displacements, and in terms of estimated total cost associated with the studied solutions in this parametric study -- it is thought that, in addition to the original solution associated with the initial tower model structure, the most indicated alternatives may be the tower models D and E.

Table 16 : Cost estimate of the tower models

Tower model	Total weight of tower (kgf) ; 1 kgf ≈ 0.01 kN	Cost estimate of tower (Euro)
Initial tower	22354.13	55885.33
Tower A	23366.31	58415.79
Tower B	29610.04	74025.09
Tower C	27087.03	67717.57
Tower D	21928.05	54820.13
Tower E	21995.66	54989.14

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