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VIBRATION ANALYSIS OF A STEEL LATTICE TOWER FOR A 24kW WIND TURBINE

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Abstract. *Most large wind turbines currently installed use self-supporting steel tubular towers. Tube towers have been used, but they represent additional manufacturing process and labor costs. Steel lattice towers are usually assembled from angle sections, with bolting used for attaching the bracing members to the legs and splicing the leg sections together. Typically the towers are square in plan with four legs, facilitating the attachment of the bracing members. One of the advantages of lattice towers is that material savings can be obtained by splaying the legs widely apart at the base, without jeopardizing stability or posing transport problems. Systems with a natural frequency below the rotor speed (1P) are classed as "soft-soft"; those with natural frequencies between 1P and nP (where n is the number of blades) are "soft"; and a frequency above nP identifies the tower as "stiff." A preliminary design of a steel lattice tower for a 24kW wind turbine has been accomplished. Among many design requirements, a free vibration analysis has been performed for a truss tower having 17.4m height and 3.6 base length. From the obtained results the first natural frequency is above the rotor speed and below 3P, where the number of blades is 3.*

Keywords*: Renewable energy; Mechanical Vibrations; Tower; Finite Elements; Wind Turbine*

1. INTRODUCTION

The main purpose of this work is to define a methodology for designing a flexible lattice tower on dynamics loads due wind generators and loads.

The aim is to design the tower according to requirements of static analysis, buckling analysis and modal analysis, for a truss tower of 17.4 m in height for a wind generator of 24 kW.

The large-scale wind turbines installed use steel tubular towers as a means of sustaining the generation package. Despite the widespread use of tubular towers, this solution would represent additional labor costs and a more sophisticated manufacturing process.

The truss towers are usually assembled from sections of square pyramids modules. One of the advantages of lattice towers is the economy of reticulated material obtained with widely splayed legs apart at the base, without jeopardizing stability or posing transport problems. These angles are limited by blade tip considerations.

A structural engineer designing of a tower for wind generator must take into account several tower loadings: extreme loads, dynamic response to extreme loads, operational loads due to steady wind, operational loads due to turbulence, dynamic response to operational loads, fatigue and stresses.

The tower of the wind generator of this project was designed in accordance with the concept of truss frames and according to the classification of flexible tower. In the analysis of structural problem, the Femap/Nastran has been used (Bussanra, 2003).

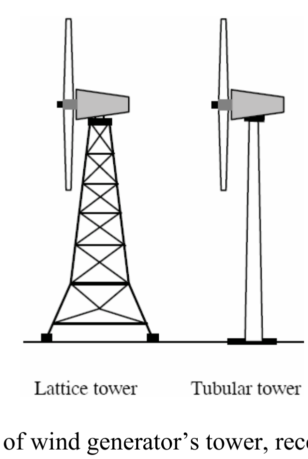

Figure 1. Main types of wind generator's tower, recently Sathyajith (2006)

2. REQUIREMENTS

Among many requirements for designing a tower, the analysis of free vibration has to be considered. The towers can be classified as "very flexible" when the fundamental frequency is below the frequency of rotation of the rotor (1P), "flexible" when the fundamental frequency is between 1P and nP (where n is the number of blades) or "rigid" when the fundamental frequency is above the NP (Burton et al 2001).

The evaluation procedure takes into account that the natural frequencies of the tower should not coincide with the frequencies of excitation. Preliminary studies indicate that the rotational speed of the rotor is 108 rpm which results in a fundamental frequency of excitation of 1.8 Hz; whereas the generator will consist of three blades, a multiple of the frequency of excitation by aerodynamic issues, is 5.4 Hz. Furthermore, a radius of 5.5 m has been adopted for the blades (Donadon et al, 2008).

For steady wind loads the effect of wind pressure to the structure is considered for static and buckling analysis. In practice, the wind can address to any side of the tower. However, for purposes of this analysis, we consider only the front face incidence and the direction forming an angle of 45 ° with the face. Obtaining a good safety margin for these angles of incidence, it is possible to ensure the strength of the tower to any other angle. To calculate the power generated by wind, ABNT 6123 (1998) is considered.

For the purpose of modeling the tower, the Finite Elements software FEMAP, v. 8.3, was adopted. For this software, NX NASTRAN solver was adopted.

3. MODAL ANALYSIS

As a first procedure for the tower design, a vibration analysis has been conducted. The content of his work is detailed in Mendes et al (2009), where a sensibility analysis was performed to some of the main variables of the problem: height of tower, profile section size and type, generator mass and reinforcements. Table 1 summarizes the performed analysis.

Table 1. Modal analysis performed to vibration sensibility analysis.

Analysis	Height	Section	Dimensions			Reinforcement	Mass
			Diameter	Longeron	Reinforcement		
	(m)		(cm)	(in)	(in)		
I	25	O	D = 1.82cm			normal	yes
II	25	L		3 x 5/16	1 1/2 x 1/4	normal	yes
III	25	L		3 x 5/16	1 1/2 x 1/4	advanced	yes
IV	25	L		3 x 5/16	1 1/2 x 1/4	normal	no
V	18	L		4 x 1/2	2 x 3	normal	yes

After a certain number of analysis, a final conception was achieved with the following performance result: 1st natural frequency equal to 3.58 Hz, according to Fig. 2, and, 2nd natural frequency equal to 5.4 Hz, according to Fig. 3. The natural frequencies of the tower are not coincident and are far from the blades frequencies.

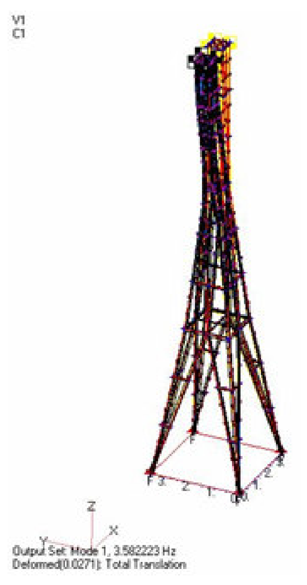

Figure 2. Bending fundamental frequency of 3.58 Hz

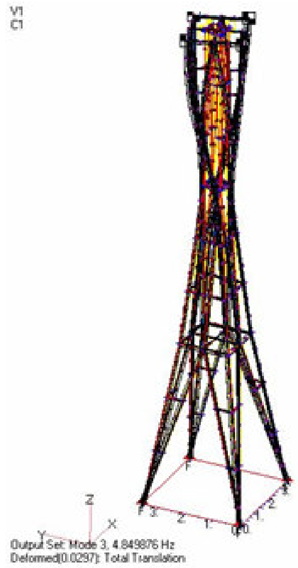

Figure 3. Torsion fundamental frequency of 5.4 Hz

Besides vibration, other verifications like static and buckling analysis must be done.

4. STATIC AND BUCKLING ANALYSIS

4.1. Theoretical development

The force of wind acting on the structure of the tower was modeled considering the ABNT 6123 (1988) standard, using the flat side of prismatic bars. Under this standard, for a corner (such as the profile used), the active forces are given by:

$$Fx = Cx.q.K.L.c \quad (1)$$

$$Fy = Cy.q.K.L.c \quad (2)$$

The coefficient C is a function of the angle of incidence of the wind in the corner (α), as outline below:

Table 2. Coefficient of proportionality

α	C_x	C_y
0^o	1,8	1,8
45^o	2,1	1,8
90^o	-1,9	-1
135^o	-2	0,3
180^o	-1,4	-1,4

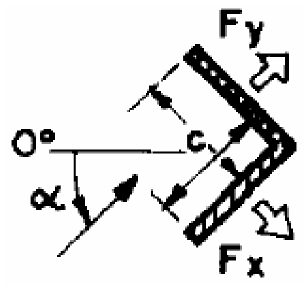

Figure 4. Angle of incidence

The parameter c is the width of the profile used. The parameter L is the length of the profile used.
K is the reduction factor due to the fact that profiles have a limit dimension.

Table 3. Reduction factor for several configurations reproduced from ABNT 6123 standard.

L/C_a	2	5	10	20	40	50	100	∞
Prismatic bars of circular section in subcritic regimen ($Re<4,2.10^5$)	0,58	0,62	0,68	0,74	0,82	0,87	0,98	1,0
Prismatic bars of circular section in regimen above crictic ($Re\geq4,2.10^5$)	0,80	0,80	0,82	0,90	0,98	0,99	1,0	1,0
Prismatic bars of plannar faces	0,62	0,66	0,69	0,81	0,87	0,90	0,95	1,0

The parameter q represents the dynamic pressure and is given by the following equation:

$$q = 0{,}613.(V_o.S_1.S_2.S_3)^2 \tag{3}$$

where V_o is the basic wind speed, which is defined as the speed of a gust of 3 s, exceeded on average once in 50 years, at 10 m above the ground in open field and background, S_1 is the topographic factor that takes into account changes in topography of the land , S_2 is the roughness and geometry factor , and, S_3 the probabilistic factor. In Tables 3, 4 and 5, reproduced from ABNT 6123 standard, these factors can be obtained.

Table 4. Roughness and geometry factor

S_2 Factor															
z (m)	Category														
	I			II			III			IV			V		
	Class			Class			Class			Class			Class		
	A	B	C	A	B	C	A	B	C	A	B	C	A	B	C
≤ 5	1,06	1,04	1,01	0,94	0,92	0,89	0,88	0,86	0,82	0,79	0,76	0,73	0,74	0,72	0,67
10	1,10	1,09	1,06	1,00	0,98	0,95	0,94	0,92	0,88	0,86	0,83	0,80	0,74	0,72	0,67
15	1,13	1,12	1,09	1,04	1,02	0,99	0,98	0,96	0,93	0,90	0,88	0,84	0,79	0,76	0,72
20	1,15	1,14	1,12	1,06	1,04	1,02	1,01	0,99	0,96	0,93	0,91	0,88	0,82	0,80	0,76
30	1,17	1,17	1,15	1,10	1,08	1,06	1,05	1,03	1,00	0,98	0,96	0,93	0,87	0,85	0,82
40	1,20	1,19	1,17	1,13	1,11	1,09	1,08	1,06	1,04	1,01	0,99	0,96	0,91	0,89	0,86
50	1,21	1,21	1,19	1,15	1,13	1,12	1,10	1,09	1,06	1,04	1,02	0,99	0,94	0,93	0,89
60	1,22	1,22	1,21	1,16	1,15	1,14	1,12	1,11	1,09	1,07	1,04	1,02	0,97	0,95	0,92
80	1,25	1,24	1,23	1,19	1,18	1,17	1,16	1,14	1,12	1,10	1,08	1,06	1,01	1,00	0,97
100	1,26	1,26	1,25	1,22	1,21	1,20	1,18	1,17	1,15	1,13	1,11	1,09	1,05	1,03	1,01
120	1,28	1,28	1,27	1,24	1,23	1,22	1,20	1,20	1,18	1,16	1,14	1,12	1,07	1,06	1,04
140	1,29	1,29	1,28	1,25	1,24	1,24	1,22	1,22	1,20	1,18	1,16	1,14	1,10	1,09	1,07
160	1,30	1,30	1,29	1,27	1,26	1,25	1,24	1,23	1,22	1,20	1,18	1,16	1,12	1,11	1,10
180	1,31	1,31	1,31	1,28	1,27	1,27	1,26	1,25	1,23	1,22	1,20	1,18	1,14	1,14	1,12
200	1,32	1,32	1,32	1,29	1,28	1,28	1,27	1,26	1,25	1,23	1,21	1,20	1,16	1,16	1,14
250	1,34	1,34	1,33	1,31	0,13	1,31	1,30	1,29	1,28	1,27	1,25	1,23	1,20	1,20	1,18
300	-	-	-	1,34	1,33	1,33	1,32	1,32	1,31	1,29	1,27	1,26	1,23	1,23	1,22
350	-	-	-	-	-	-	1,34	1,34	1,33	1,32	1,30	1,29	1,26	1,26	1,26
400	-	-	-	-	-	-	-	-	-	1,34	1,32	1,32	1,29	1,29	1,29
420	-	-	-	-	-	-	-	-	-	1,35	1,35	1,33	1,30	1,30	1,30
450	-	-	-	-	-	-	-	-	-	-	-	-	1,32	1,32	1,32
500	-	-	-	-	-	-	-	-	-	-	-	-	1,34	1,34	1,34

Table 5. Probabilistic factor

Group	Description	S_3
1	Buildings which total or partial collapse may affect the safety or capability to help the people after a destructive storm (hospitals, barracks for firefighters and arm headquarters, communications, centrals, etc.).	1,10
2	Buildings for hotels and residences. Buildings for commerce and industry with high occupancy factor.	1,00
3	Buildings and industrial facilities with low occupancy factor (warehouse, silos, rural buildings, etc.).	0,95
4	Fences (roof, windows, fence boards, etc.).	0,88
5	Temporary buildings. Structures of Groups 1 to 3 during construction.	0,83

4.2. Loading on tower

Based on the theoretical development, one may identify the values of the relevant parameters to calculate the force in the tower and proceed with the static and buckling analysis. However, two points must be highlighted:

- In the analysis of the tower, one must consider the effect of axial load produced by the wind blades on the crane. Aerodynamic studies (Donadon et al, 2008) indicate that this value is 4900 N for a critical situation. This load was assumed to be acting at a point 60 cm above the top of the tower.

- This is an analysis of strength and buckling and the main purpose is checking the safety margin of the model according to these requirements. At this point one may anticipate that the vibration requirement has been the most critical, and any overestimated load will not play an important issue in this analysis.

For the calculation of wind pressure, the horizontal profiles of the tower are smaller and neglected(which have a small area in comparison to vertical). For vertical profiles, c = 4 inches and L = 5.8 m were adopted. Besides, the following values were taken into account:

$V_o = 30 \text{ m/s}$.

S_1 = 1 (considering that the tower will be installed on a local level or a little rough).

S_2 = 1.06 (assuming that the terrain of the site in question falls under Category II and of 17.4 m being the total height of the tower, in Class A.

S_3 = 1 (good degree of security).

From the assumed values and using Eq. 3, we can calculate the dynamic pressure:.

$$q = 0,613.\left(V_o.S_1.S_2.S_3\right)^2$$
$$q = 0,613.\left[(30).(1).(1,06).(1)\right]^2$$
$$q = 619,9 \text{ N/m}^2$$

Thus, replacing the estimated value and using Eqs. 1 and 2, we have:

$$F_x = C_x.q.K.L.c$$
$$F_x = C_x.(619,9).(1).(5,8).(4.0,0254)$$
$$F_x = 365,3.C_x \text{ (N)}$$

$$F_y = C_y.q.K.L.c$$
$$F_y = C_y.(619,9).(1).(5,8).(4.0,0254)$$
$$F_y = 365,3.C_y \text{ (N)}$$

For the analysis focusing the head wind on the tower, the angles involved are α = 45 and α = 135. Each profile is then analyzed according to the incident angle of the wind and its coefficient (according to table 2) replaced in the equation above.

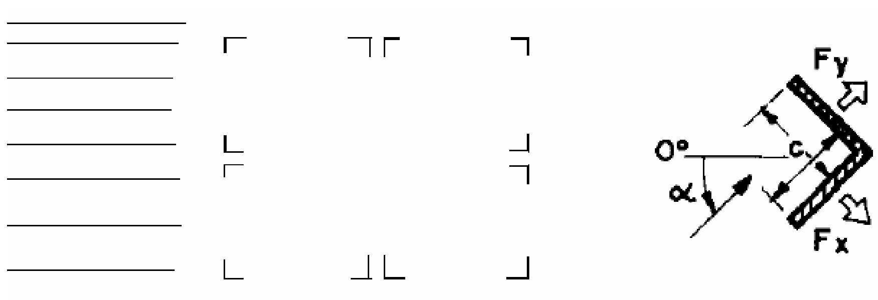

Figure 5. Top view of wind profiles in front

In the case of wind forming an angle of 45 with the front face of the tower, the angles involved are 0o = α, α = 90 ° and α = 180. Here again we must examine each profile separately, and according to the angle of incidence on that profile, their relationship coefficient.

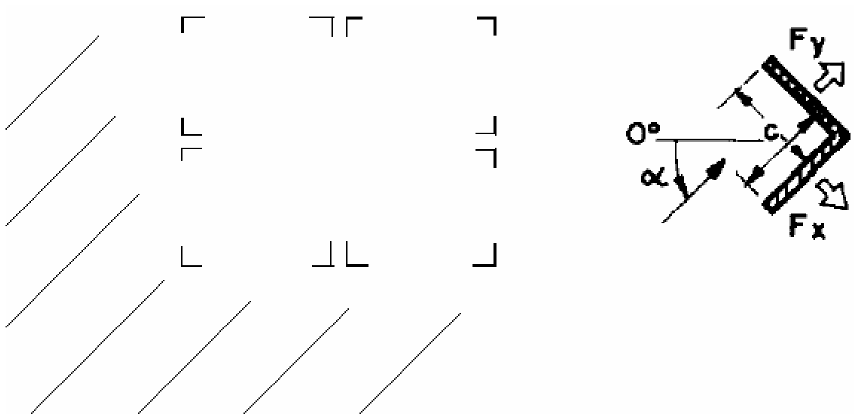

Figure 6. Top view of the profiles for wind with an incidence of 45

5. RESULTS

According to item 4, two cases were assumed according to the load distribution:

- frontal incidence; and,
- oblique incidence (45).

Based on these types of loading, the static and buckling analysis was performed.

5.1. Frontal case

The next figure displays the stresses produced by frontal loading on the tower.

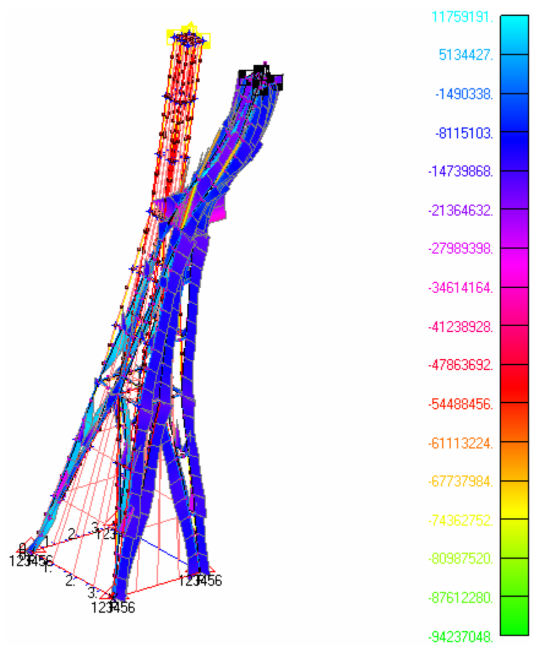

Figure 7. Maximum stress of 94,106 N/m2

In the analysis of buckling in frontal loading case, we obtain the following configuration:

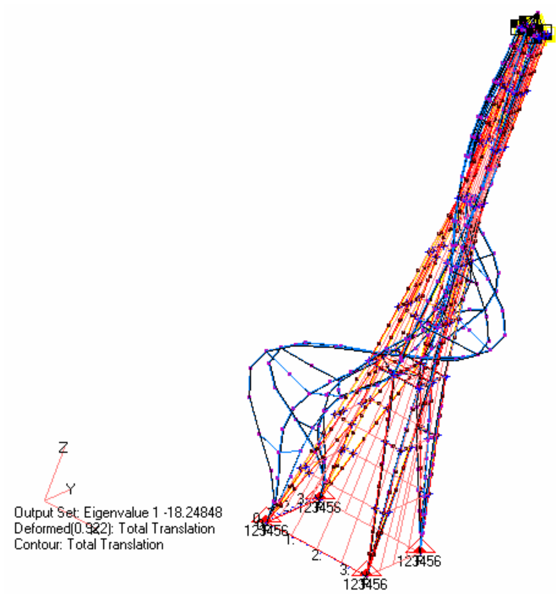

Figure 8. 1st buckling mode, where critical load 18 times greater than the active

5.2. Oblique incidence case

The next figure displays the stresses produced by oblique loading on the tower.

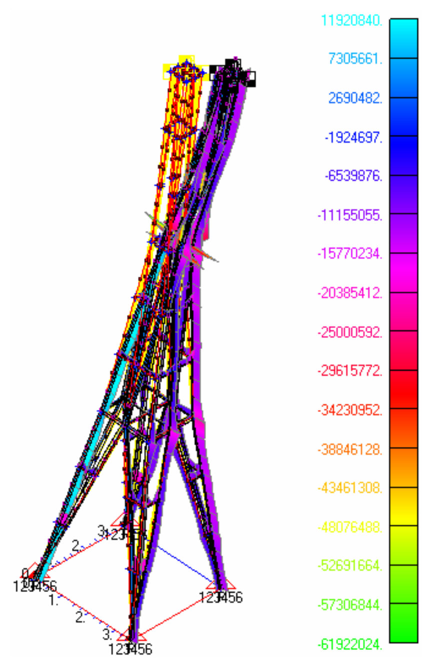

Figure 9. Maximum stress of 62,106 N/m2

In the analysis of buckling for the second case, we obtain the following first buckling configuration:

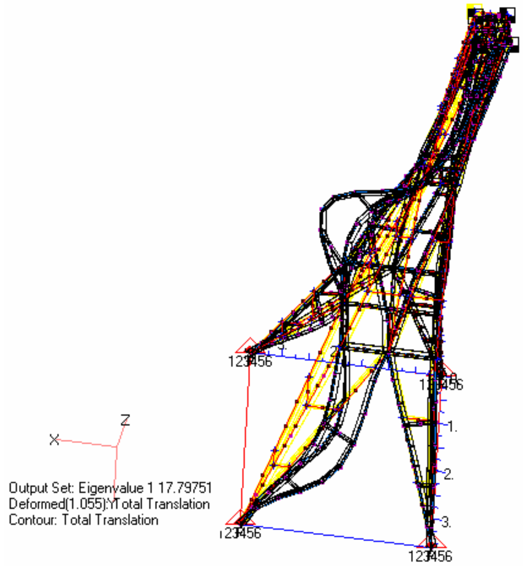

Figure 10. 1st buckling mode, where critical load is 18 times greater than the active

6. CONCLUSION

Recently, Mendes et al (2009) have validated the vibration analysis of the presented tower. For the sake of safety and model validation, the same tower has been examined according to the static and buckling criterions. Taking into account a gust wind velocity of 30 m/s, the present results reveal a safety margin of 18 for the buckling analysis and small maximum stress of 94.106 N/mm2 compared to the steel yield strength of 300N/mm2 for the static cases.

According to these results, the vibration criterion of analysis has been revealed as the most critical in this type of design. Nevertheless, one may not neglect the static and buckling analysis, and possibly in other cases they might reveal a more important sensitivity.

7. ACKNOWLEDGEMENTS

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