RESONANCE OF STEEL WIND TURBINES: PROBLEMS AND SOLUTIONS

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Abstract

This paper deals with an experimental and numerical dynamic characterization of two steel wind turbines currently located in Italy. The considered steel turbines are onshore and were originally built more than 20 years ago with 65 meters of height (from the ground level to the hub). Five years ago, they were unmounted and reassembled in the present location. During this phase, due to local and national environmental prescriptions, they were shortened to about 45 meters (hub height). Unfortunately, this procedure generated a huge problem in their structural behaviour: the fundamental frequency of vibration of both the "shorter" steel turbines become close to the working frequency of their rotors, causing a resonance problem.

To evaluate exactly the turbine frequencies and modal shapes, structural monitoring involving several accelerometers has been implemented. Then, numerical finite element models (FEM) have been developed to investigate the influence of the soil-structure interaction (SSI) on the dynamic behaviour of these structures.

Finally, two practical solutions to overcome the resonance problem are proposed and discussed.

Since the investigated problems can arise for any worldwide wind turbine, the proposed solutions are of general validity and can be extended to many other different cases.

Keywords: *steel wind turbines, modal identification, numerical calibration, soil-structure interaction (SSI), resonance*

1. INTRODUCTION

Climate change and environmental degradation are a huge threat to Europe and worldwide. To overcome these challenges, the European Green Deal has been recently developed with the main aim to promote the efficient use of resources by moving towards a clean and circular economy [1]. Energy production from renewable sources is the challenge on which a good portion of modern scientific research is focusing [2]. This is mainly done to reduce pollution resulting from extreme and extensive use of oil. In this direction, obtaining electricity by exploiting the force of the natural wind is an excellent eco-sustainable and very effective solution in terms of costs, environmental impact and production. To this aim, offshore and onshore steel wind turbines are the most used solutions in Europe [3]. In the last 50 years, a great number of wind parks have been erected through Europe, giving a huge boost to the growth of renewable energy production. A study conducted in 2016

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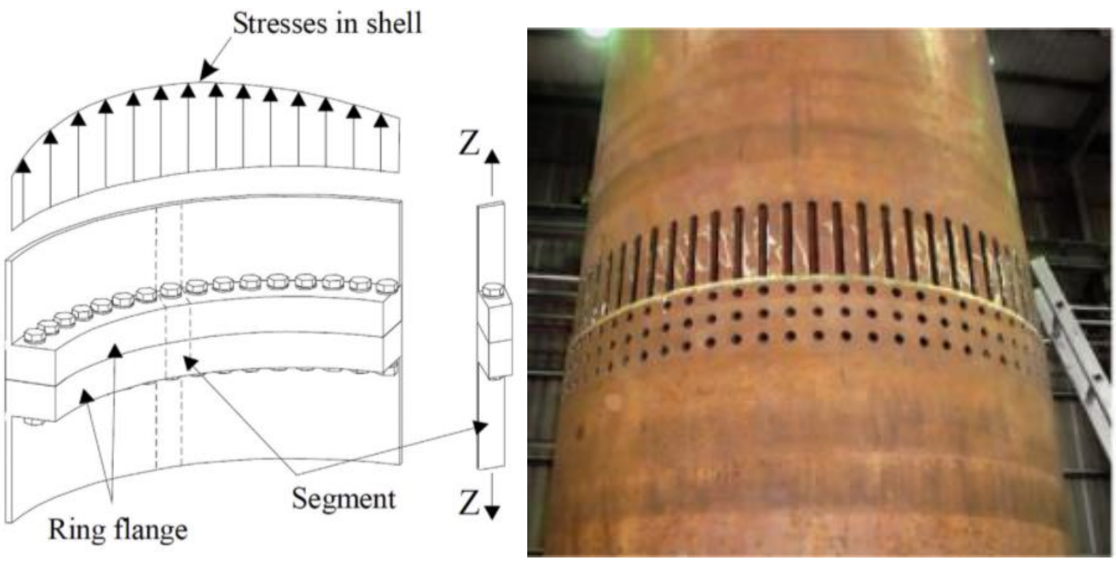
declares that *341.320 wind turbines were installed across the world and globally more than 637.000.000 tonnes of carbon emissions were avoided* [4]. Moreover, additional to the well-known onshore and offshore wind parks, small scale horizontal and vertical wind turbines are now also being installed in private properties [5]. The onshore steel turbines are generally realized using a tubular steel column with different diameters along with the height while the offshore structures are a mix solution between lattice trussed and tubular columns [6,7].

The main objective of the owner of the wind farm is to produce the maximum quantity of energy in minimum time. However, national and local laws impose dimensional limitations to the steel turbines, in terms of diameter and total height. Depending on the zone in which they are installed, the height of a single column can range from 15 to 80 meters. Wind turbine columns are realized with different circular steel segments having different diameters along with the height. The connection between these circular elements is generally bolted (Fig. 1) and can be realized with friction or preloaded connections [8]. As an example, the geometry of the most common welded ring flange connection is presented in Fig. 1a. It can be observed that flanges are connected with high-strength preloaded bolts (class 10.9) and therefore is of paramount importance to assure a more than good contact between surfaces by using flatness rig flanges. According to EN1090-2 [9], the construction tolerances are very limited, and this probably represents the most relevant constructional difficulty of this connection system. Between the models specifically developed during the years for the study of these connections, the Petersen model (updated by Seidel [10]) is the one most used. One of the main objects of past research was the improvement of the fatigue resistance of these connections, which generally governs the global design of the steel towers. For the fatigue, reference is made to detail category 71, EC3 1-9 [11]. The improvement of these details brings also to an increase in the tower heights and hence to a boost in energy production. To this aim, several European projects have been developed [12,13] on bolted friction connection (Fig. 1b) used as the solution to improve the fatigue behaviour. They have been extensively analysed both experimentally and numerically, under monotonic and cyclic conditions. Pavlovic et al. [14] showed a detailed comparison between both connections showing the improvement of the second solution in term of fatigue resistance



a) b)

Figure 1. Example of different typologies of connections on steel wind turbines: a) preloaded and b) friction type, ref. [6,13].

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Wind turbine foundations are very different case to case, depending on the considered structure. For offshore structures monopiles, jackets, tripods or gravity-based foundations are the most commonly used [15]. On the other hand, onshore steel turbines realized employing hollow circular steel profiles presents a unique huge reinforced concrete plinth with eventually the adjunction of piles (Fig. 2). In this case, it can be underlined that these enormous concrete foundations are particularly hard to dispose of once the steel tower is removed. Besides, the soil-structures interaction (SSI) is rarely properly considered in the design.

Figure 2. Example of concrete foundation for onshore steel turbine: reinforcement rebars arrangement (left) and concrete block (right).

Despite a great engineering experience is required in the design of these huge and complex structures, some collapse happened in recent years, in Italy and other parts of the world [16]. It can be estimated that the collapse of the tower leads to an average economic cost between \$500.000 and \$5.000.000, depending on the turbine typology. The other parts (hub, electrical components and blades) can be repaired, and the energy production can continue. As discussed, main causes of these collapses are electrical problems on rotors that caused fire, insufficient strength of bolts, poor bolt quality control during construction or collapse of the connections due to maximum fatigue life (Fig. 3). Moreover, the rotor and the steel turbines resonance generate huge and dangerous displacements that can yield to collapse. More in detail, in ref. [17] a complete overview of 48 tubular wind tower collapse incidents that happened over the last 40 years is presented, discussing damage typology and causes. The proposed investigation indicated that malpractices, defects on material, and accidental loads induced by typhoons and storms are the most common reasons for failure. An interesting graph is proposed in Fig. 3 that is the bathtub curve of the overall failure rate of wind turbine components over time. Three typical stages have been reported: early-life, operational and wear-out-stages. Faulty construction and great material defects bring to tower failure generally located in the “infant failure” part. During the normal operational life, failure rate became quite low: in these cases, defective operation, resonance problems, and improper maintenance can bring to collapse. The last part is related to the collapse due to inadequate maintenance and fatigue effects (mainly failure of connections). It has been demonstrated that, generally, the typical steel turbines show the first sign of a failure after 15/20 year of operation. For this reason, after 15 years, the steel turbine towers are generally dismantled and this is not fully consistent with the idea of the European green deal requirements. To overcome this problem, structural designers and codes should focus more attention

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on the increase of performance of parts and components such as connections and towers. On the other hand, constructors must improve quality controls and maintenance.

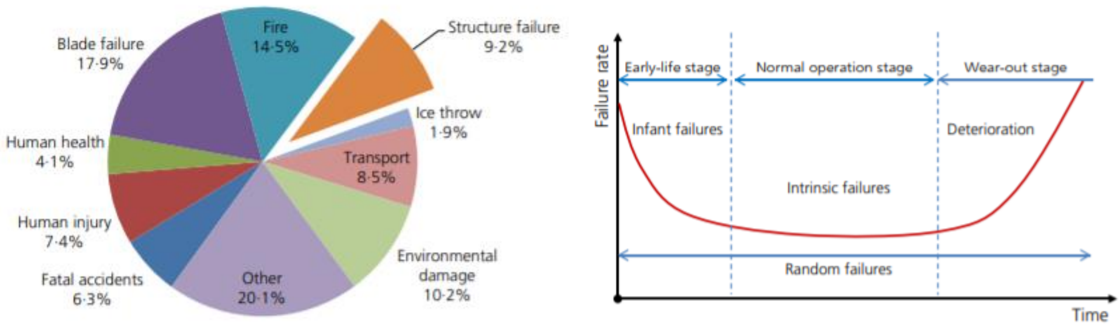


Figure 3. Failure type distribution between 1990 and 2016 and bathtub diagram [17].

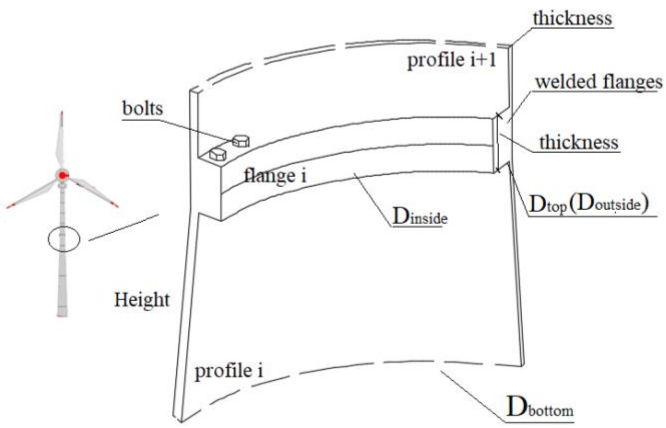
In the following, two onshore steel wind turbines were considered and analysed. The main peculiarity of these turbines is that they were located in Europe for 20 years then disassembled and rebuilt in Italy with a different height. In this current situation, large top displacements have been measured once the rotors have been turned on. These uncommon and unexpected displacements resulted in a complete block of the system for safety reason. To overcome this problem and understand the dynamic behaviour of these turbines, a complete *in-situ* experimental modal identification has been executed 4 years ago by an Italian engineering office. The main result pointed out from this study was related to a resonance problem: the first fundamental frequency of both the turbines is close to the working frequency of their rotors. These experimental activities are the starting point of the paper: after a discussion on the obtained results, advanced numerical models have been developed considering also the influence of SSI. Finally, to remove the resonance problem, two different practical solutions have been presented and discussed: i) application of a tuned mass damper (TMD) system and ii) rising the tower to its original height. While the latter solution must be consistent with local environmental laws, the former could always be applied. In literature, large research has been devoted to the use of TMD in steel turbines (mainly for offshore typologies) [18-20] but generally only pure theoretical models have been presented without indications about costs and realization. Instead, both the solutions proposed in the present paper have general validity and can be extended to every wind tower facing resonance issues.

1.1 DESCRIPTION OF THE CONSIDERED TURBINES

The wind towers were built 40 years ago. These wind turbines do not have any system for the number of revolutions reduction and are equipped with a crankshaft which directly connects the generator to the propeller rotor consisting of three blades. The asynchronous generator has a variable number of revolutions, characterized by a 2.5 m diameter. The maximum power is approximately 500 kW at a rotation speed of 38 rpm. All the connections have been realized using welded ring flanges with preloaded high-strength bolts like the one showed in Fig. 1a. The blades rotor diameter is about 40 m. The initial hub design height of the supporting tower was 65 m. The foundation is a single huge reinforced concrete cylinder (13 m of diameter by 3 m of height) plus 16 concrete piles (0.8 m of diameter for a length of 10 m). From the geotechnical inspection, it has been assessed that the soil was composed mainly of clay.

To understand how the turbines were originally designed, Table 1 presents the height, the diameter and the thickness of each tubular steel segments regarding the original (old) configuration. Pieces not connected with flanges (e.g. P1-P2) are directly welded together. A tubular piece of steel has been embedded inside the concrete foundation and then connected with P1 using a ring flange. All the tapered pieces present variable diameters along with their height, decreasing from the bottom (D_{bottom}) to the top (D_{top}). The external diameter of the flange ($D_{outside}$) is equal to the external diameter (D_{top}, D_{bottom}) of the tubular steel segment where the flange is welded to.

Table 1. The original configuration of the wind turbine under study.



Profile [n°]	Height [m]	D_{bottom} [mm]	D_{top} [mm]	Flange [n°]	thickness [mm]	$D_{outside}$ [mm]	D_{inside} [mm]	Bolts
				F1 (P1-base)	60	3228	2708	160 M39
P1	3.5	3228	3107	-	22			
P2	17.5	3107	2150	-	20			
				F2 (P2-P3)	100	2150	1920	160 M39
P3	17.5	2150	1985	-	20			
P4	3.5	1985	1675	-	18			
				F3 (P4-P5)	75	1675	1420	104 M39
P5	7.0	1675	1565	-	18			
P6	3.5	1565	1495	-	16			
P7	3.5	1495	1425	-	14			
P8	3.5	1425	1355	-	12			
P9	3.5	1355	1200	-	10			
				F4 (P9 – rotor)	40	1200	1000	72 M24
total	63.0							

The turbines were removed and re-installed in Italy after 20 years of service in the first location. Due to local and national environmental standards, the turbines power was downgraded to 200 kW and their height was reduced to about 45 m (see Table 2 with the current geometrical characteristics). This has been obtained through the realization of a new piece (namely P3b in Table 2) characterized by the following dimensions: height = 3.5 m, D_{bottom} = 2.15 m, D_{top} = 1.675 m and thickness = 20 mm which has been installed replacing the original P3 and P4 pieces (Table 1). The bottom and top flanges

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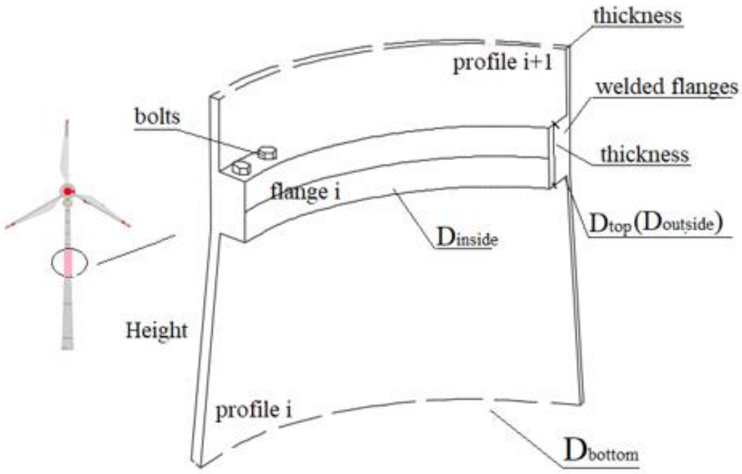
of the new piece P3b present the same characteristics, in terms of dimensions, bolts diameter, and bolts number, of the original flanges F2 and F3.

Table 2. The current configuration of the wind turbine under study.



Profile [n°]	Height [m]	D_{bottom} [mm]	D_{top} [mm]	Flange [n°]	thickness [mm]	$D_{outside}$ [mm]	D_{inside} [mm]	Bolts
				F1 (P1-base)	60	3228	2708	160 M39
P1	3.5	3228	3107		22			
P2	17.5	3107	2150		20			
				F2 (P2-P3b)	100	2150	1920	160 M39
P3b	3.5	2150	1675		20			
				F3 (P3b-P4)	75	1675	1420	104 M39
P4	7.0	1675	1565		18			
P5	3.5	1565	1495		16			
P6	3.5	1495	1425		14			
P7	3.5	1425	1355		12			
P8	3.5	1355	1200		10			
				F4 (P8 – rotor)	40	1200	1000	72 M24
total	45.5							

Since in both the turbines, the profiles have been realized with a grade S235 steel (yielding tension f_y = 235 MPa), they belong to class 4, according to the prescriptions of EN1993-1-1 [21]:

$$\frac{D_{bottom}}{thickness} > 90\frac{f_y}{235} = 90 \qquad (1)$$

From a practical point of view, profiles belonging to class 4 of EC3 are made by slender plates in which local buckling phenomena occurs if a compression state is present (caused by pure compression or bending). Therefore, the cross-section can neither rely on the plasticity nor reach the elastic resistance limit. In all the structural safety checks, the cross-section properties used in the verification equations (i.e. gross area and section moduli) must be reduced by evaluating the effective properties (effective area and effective section elastic moduli), by following the iterative procedure described in EN1993-1-6 [22]. For the verification checks, two methods are allowed: i) Reduced stress method, whereas the main input parameters are the buckling of the cylindrical shell and the squash load

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determined by using Annex A of the same code. This method is based on reduction of design resistance of membrane stress component (meridian, circumferential and in-plane shear stresses must be used in the design check; ii) design by numerical geometrically and mechanically non-linear analysis accounting for initial imperfections (GMNIA). Generally, in this method, the initial imperfection arises from an elastic buckling analysis (LBA). Despite the second one is the most refined procedure, there are some aspects associated with the calibration of the numerical model that must be always considered with great attention.

It can be easily verified that all the required structural checks were satisfied in the original configuration, including also the fatigue life check of the connections. Furthermore, after the shortening process, the state of stress became even better on the tower. Despite structural checks are satisfied, once the shaft rotation value becomes greater than 28 rpm (operational rotation condition to guarantee an optimum level of energy production), both towers showed severe oscillations that caused the trigger of the safety systems (lock of the rotor) based on the output of the instruments installed on the hub. It has been assessed that the oscillations were independent of the environmental conditions. Thus, it can be excluded a dependence of this phenomenon from the wind intensity and directions. By analysing the preliminary data, the complete lock was not due to the absolute value of the displacement reached on the top but it was due to the repeated oscillation (change in sign of the top displacement) increasing over time.

2. EXPERIMENTAL CAMPAIGN

An experimental modal analysis campaign was carried out using uniaxial piezoelectric seismic accelerometers PCB 393A03 type (sensitivity 1000 mv/g, range +-5g). Considering a preliminary theoretical modal analysis six measurement points have been identified along with the steel tower height, see Fig. 4.

In addition to the six measuring points, some additional instruments have been installed on the two turbines:

- an accelerometer was installed on the Turbine 1 inside of the hollow shaft, at the rear bearing, which supports the generator;
- in Turbine 2, a biaxial measuring point (two accelerometers) was installed half of the upper segment, aimed at better characterizing the modal shapes.

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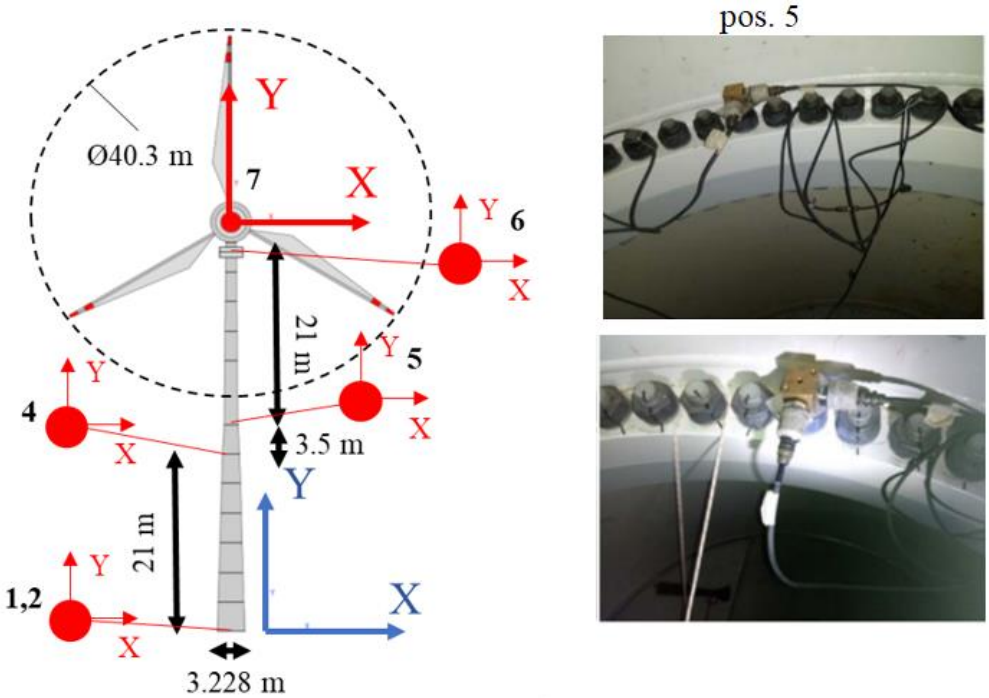

Figure 4. Dimensions of the considered turbines and measuring points disposition.

To cover all the measurement points, several measurement sessions were conducted with different arrangements of the available sensors. To perform modal identification, where the dynamic excitation is not known, Operational modal analysis (OMA) technique [23] should be used. In this case, we assume that the stress on the investigated structure can be approximated with white noise. However, the main part of the dynamic load is composed of several components, typically from artificial and/or natural sources of vibration (traffic, industrial plants, wind, seismic microtremor, etc.) present around the structure. Since the input is unknown, it is possible to analyse only the response of the structure (modal output-only analysis). The analysis of the acquired data is based on the study of the Power Spectrum function, Crosspower-spectrum of two signals, one of which is taken as a reference between all the collected data sets. Any peak in its graphical representation indicates the presence of predominant frequencies in the signals. This function can be decomposed into series as a combination of the frequency modal parameters, damping and deformation. In the specific case under consideration, two different working conditions have been considered:

- Wk1. Steel wind turbines under environmental conditions with rotor at rest;
- Wk2. Steel wind turbines at the maximum rotation allowed by the wind, condition that must guarantee a good energy production.

Main results and parameters considered during the tests are reported in Table 3.

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Table 3. Results associated with measurement conditions 1.

Work condition	Turbine	speed rotor [rpm]	Speed rotor [Hz]	Speed wind [m/s]	I-II mode Freq. [Hz]	I-II mode Damp [%]	III-IV mode Freq. [Hz]	III-IV mode Damp [%]
Wk1	1	0.0	0.0	3.0	0.61	3.1	3.9	1.2.
	2	0.0	0.0	8.0	0.62	3.2	3.8	1.5
Wk2	1	32.0	0.53	4.0	0.55	-	4.0	3.2
	2	34.0	0.57	10.0	0.60	2.8	4.0	1.7

It can be highlighted that both the natural frequencies and the damping coefficients are the same for the two structures. This can be expected for dynamic systems that work in the elastic range with viscous damping very close to zero. In addition, this confirms that the two systems work with the same masses and stiffness, i.e. the foundation systems bring to the same dynamic behaviour. Furthermore, in Wk2 the main frequencies of the structures are almost equal to those of the rotor shaft, see Table 3. Thus, the steel wind turbines are therefore exposed to a resonance problem. The condition of resonance leads to an increase in terms of displacements and to a higher fatigue load.

The eigenmodes shapes, practically coincident for the two structures, are reported in Fig. 5 for Turbine 1. The first vibration mode shape represents both the 2 orthogonal fundamental modes having almost identical frequencies The deformed shape is the classic one for an inverse pendulum with a rotational mode of the tower head that follows an elliptical trajectory on the horizontal plane. Finally, the Cross-spectra density is proposed in Figure 6b to underline the prevalence of the first mode with respect to the others. In fact, the greatest content of energy can be observed around the 0.6 Hz frequency (highlighted in dark-red colour). The other non-negligible content of energy is close to about 4.0 Hz.

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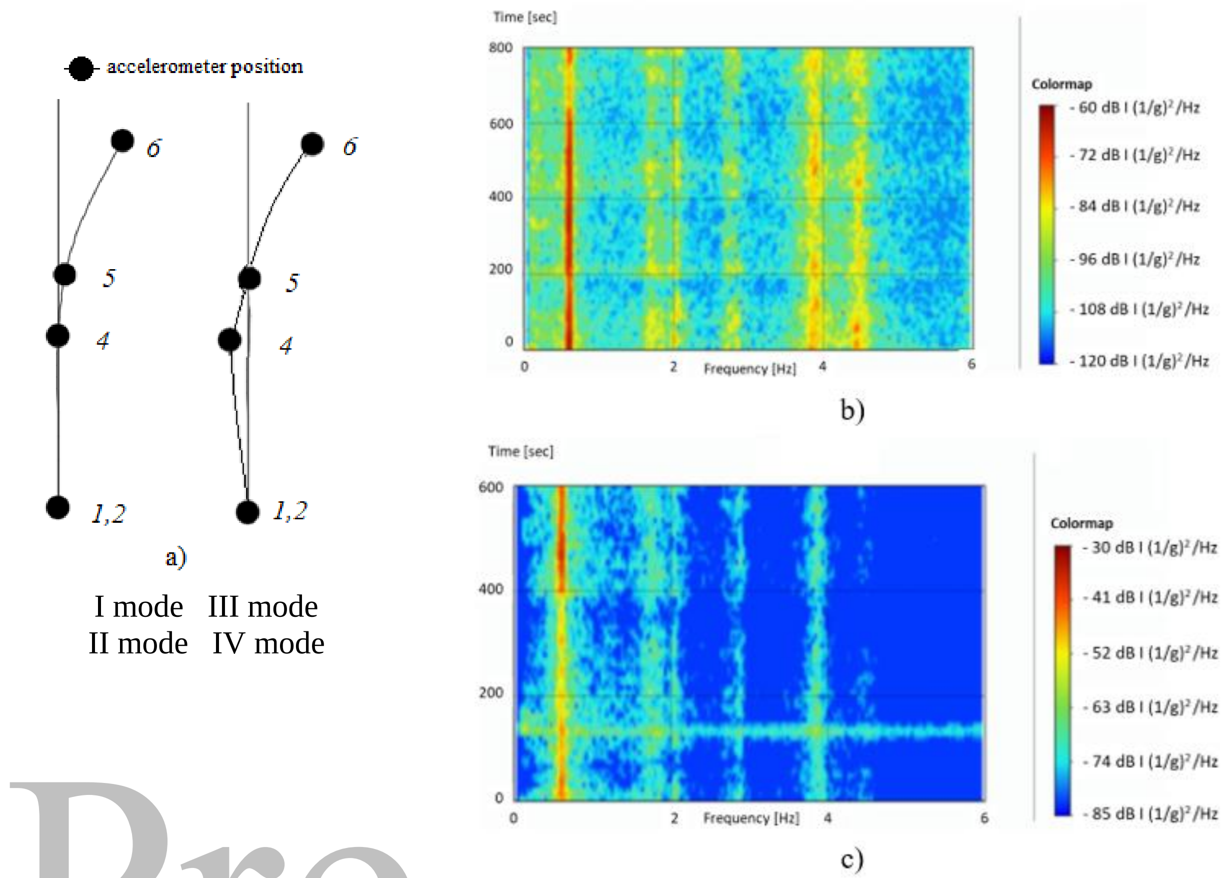

Figure 5. a) modal shapes, b) cross-spectra of Turbine 1 and c) cross-spectra of Turbine 2.

According to DNVGL-ST-0126 standard [24], each natural frequency of the tower must be different than the one of the rotors of at least 5%, that means:

$$\frac{f_R}{f_{0,n}} \leq 0.95 \quad \text{or} \quad \frac{f_R}{f_{0,n}} \geq 1.05 \tag{2}$$

where f_R is the frequency of the rotor and $f_{0,n}$ is the *n-th* natural frequency of the tower. It can be noted from Table 3 that for the first vibration mode the conditions presented in equation (2) are not satisfied.

Finally, looking at the measurements performed at the foundation level (accelerometer n°1), in Wk2 Turbine 1 shows a maximum acceleration intensity value equal to 1.8 mg, 4.5 times greater than the one recorded in Wk1. Otherwise, for Turbine 2 in Wk2, the maximum acceleration is equal to 1.1 mg which is 3 times greater than the one recorded in Wk1. For both towers, there is a non-negligible increment of the base acceleration when the rotor is in working condition.

The study of the dynamic behaviour of the two wind towers showed that the fundamental frequency of about 0.6 Hz, very close to the working frequency of the rotor. This fact exposes the structures to resonance phenomena which can be triggered and amplified by even small variations of initial conditions: little imbalances in the masses and angles of attack of the blades, misalignment in the control bodies of the nacelle orientation, dissymmetry in the structure of the towers, asymmetries in the interface between foundations and structures, peculiar turbulence phenomena due to the

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orography or the presence of other structures for particular wind directions, differential settlements of the foundation plane. Furthermore, additional problems can produce the lock of the system for security reasons.

3. NUMERICAL MODEL CALIBRATION

From a structural standpoint, the wind tower can be simplified as a cantilever fully fixed at the base, having a mass concentrated at the free end and generally with a variable cross-section along its length (Fig. 6). With a more refined calculation, the interaction of the structure with the ground can be considered by assuming a set of springs, whose stiffness can be calculated with a harmonic analysis [25], instead of a perfect fixed base constraint.

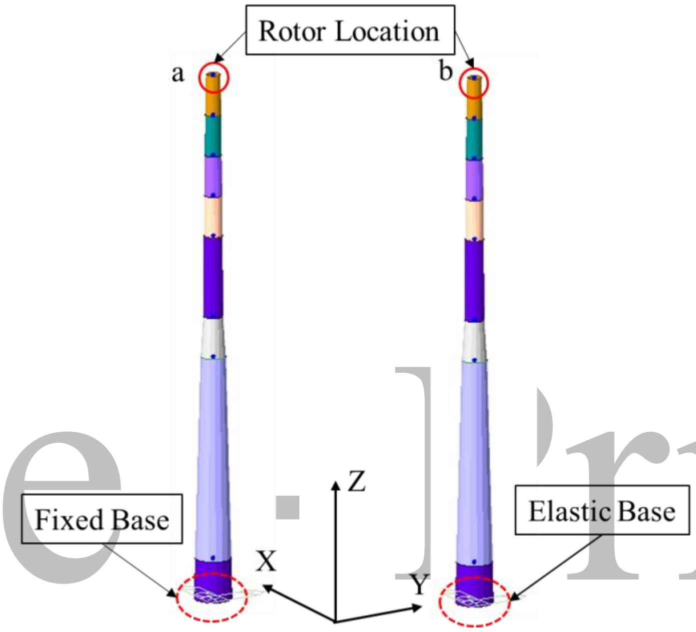

Figure 6. Static scheme of a wind tower cantilever: a) fully fixed and b) elastic base

The Finite Element Models (FEM) are implemented using a commercial Finite Element Analysis Package (MIDAS Gen [26]), where the steel tower is represented through tapered beam elements, one for each piece reported in the previous Table 1, while the rotor and the generator are considered as an equivalent nodal mass located at the top of the tower with a total weight load F_z = 290 kN. Furthermore, an additional moment equal to M_y = 378 kNm, due to the eccentricity of the rotor and the generator with respect to the tower centre of gravity, is applied at the top node of the tower model. The mechanical properties of materials adopted for the numerical models are reported in Table 4 where E is the Young modulus, γ is the unit weight, υ is the Poisson modulus, f_y is the yield strength and f_u is the ultimate strength. The total weight of the structure, considering the presence of both the rotor and the generator, is 712.6 kN. These values were assumed by following European code prescriptions: no coupon tests have been executed.

Table 4. Materials mechanical properties of the tower.

	Steel	E [GPa]	γ [kN/m^3]	υ [-]	f_y [MPa]	f_u [MPa]
Profiles	S235	210	76.98	0.3	235	360
Flanges	S355	210	76.98	0.3	355	510

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To investigate the influence of the underground soil on the modal parameters of the turbines, different models were developed:

- models F1 and F2 are the models of the two turbines with a fully fixed base (Figure 7a). This represents the more rigid option, and it is the one usually adopted in routine design;
- models S1 and S2 are the models of the two turbines also considering the elastic behaviour of the soil (Figure 7b) [27-29]. The stiffness of the rotational springs located at the base of the wind turbines can be assumed equal to $5 \cdot 10^6$ kNm/rad, associated to a quite rigid soil-foundation system. The foundation system is as rigid as possible to completely avoid rocking effects.

The modal analysis results obtained with these models are reported in Table 5, where the frequency and the percentage of the participant mass are presented.

Table 5. Main frequencies (f) and participant masses in percentage (m%) from the analyses.

	I-II mode		III-IV mode	
model	f [Hz]	m%	f [Hz]	m%
F1	0.62	50.3	4.22	16.1
F2	0.55	50.1	3.44	15.8
S1	0.59	51.1	4.02	15.7
S2	0.52	51.1	3.39	15.8

The first and the second mode, as well as the third and the fourth one, are practically identical one to another but showing orthogonal deformed shapes (i.e., I-III mode in x direction and II-IV mode in y direction). For the proposed models the influence of the SSI is quite low: the modal shapes remain exactly the same, the ones associated to the inverse pendulum, and the variation on the frequencies is always lower than 6%.

To understand which model better fits the experimental results, the mode shapes previously obtained from the OMA were compared with the numerical ones via the Modal Assurance Criterion (MAC) [30]. The MAC coefficient correlates two sets of modal vectors and it can be defined as:

$$MAC\left(\emptyset_{A,k}, \emptyset_{B,k}\right) = \frac{(\emptyset_{A,k}^T \emptyset_{B,k})^2}{(\emptyset_{A,k}^T \emptyset_{A,k})(\emptyset_{B,k}^T \emptyset_{B,k})} \tag{3}$$

where $\emptyset_{A,k}$ is related to the data set A (experimental) and $\emptyset_{B,k}$ the related to the data set B (numerical). The MAC coefficient ranges from 0 to 1: a value close to 0 indicates orthogonal eigenvectors, i.e. not correlated vectors, while 1 implies perfect correlation. In general, when MAC > 0.80 the match is good while for MAC < 0.40 the match is extremely poor.

Moreover, to compare the differences in terms of frequencies, the parameter D_F is defined [31] as:

$$D_F = \left|\frac{f_{FEM} - f_{exp}}{f_{FEM}}\right| \cdot 100 \ [\%] \tag{4}$$

where f_{FEM} is the frequency value obtained from finite element model and f_{exp} is the experimentally observed one. The lower is D_F and the better is the agreement between the compared frequencies. In table 6, the results in terms of both MAC and D_F are reported. It can be noted that, since the participant mass is mainly involved in the first two modes, only these two modes were considered.

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Table 6. Evaluation of the numerical match (E = 210GPa).

Predominant direction	F1		F2		S1		S2	
	MAC	D_F	MAC	D_F	MAC	D_F	MAC	D_F
I-II mode	0.950	11%	0.953	9%	0.964	7%	0.959	15%
III-IV mode	0.911	6%	0.925	16%	0.977	1%	0.932	18%

In all the considered cases, the MAC value detects a good correlation between the numerical and experimental modal shapes: it results always greater than 0.950. Also, the D_F value underlines a good match between all the numerical models and the experimental ones. The differences between the fully fixed base and the model with elastic supports are limited.

Finally, since no experimental data are available on the effective value of Young modulus (E), the influence of E on the numerical results was investigated by considering a variation of approximately ±5%, i.e. the analyses were also repeated by assuming the values E= 200000 MPa and E = 220000 MPa. It is worth noting that, according with both research and design practice, E is usually assumed as the nominal value recommended by Standard Provisions (i.e. E=210000 MPa). Despite that, sometimes the variability of E for structural steel cannot be neglected for practical design purposes [32], owing to the production processes as well as to small differences on the chemical components. As a consequence, E variation is expected to reflect directly for which concerns the stiffness and hence the modal responses. The numerical results obtained with these additional analyses showed that the variation of frequencies is not directly proportional on the considered modulus of elasticity: for fixed models with E = 200000 MPa the fundamental frequency variation is of about 3% while for S-models is about 2.5%. For the higher modes, the variation is more significant up to 8%.

4. MITIGATION OF VIBRATION

In order to reduce the effects of the resonance phenomenon (alternate variation of the top displacement, which increase in time) described in the previous Sections, two different solutions have been considered: (i) the installation of a damping device and (ii) the modification of the steel tower height, resulting in a change of the eigenfrequencies. In literature, the use of active or passive mass dampers is described [33,34]. In this work, as a first solution, a TMD is considered. The damper is schematized in the Finite Element Model as a nodal mass linked to the top of the tower by a linear spring-damper element (Fig. 7) [35]. The TMD stiffness ratio and the damping coefficient are calculated considering [36] only the wind turbine configuration characterized by the elastic base. Table 7 summarizes the fundamental parameters of the TMD considered in this work, where ω_{TMD} is the tuned mass damper frequency, m_{TMD} is the mass of the TMD, and K_{TMD} and C_{TMD} are the equivalent stiffness and damping of the TMD, respectively.

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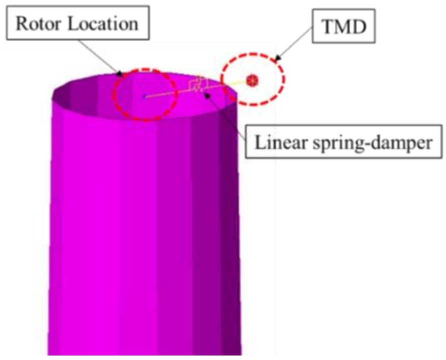

Figure 7. TMD modelling schematization.

Table 7. TMD parameters.

ω_{TMD}	0.6268 [1/s]
m_{TMD}	14.46 [kN]
K_{TMD}	22.85 [kN/m]
C_{TMD}	37.71 [kNs/m]

The mechanical properties of the tower are the same reported in the previous Table 4. To evaluate the mitigation effects on top displacements due to the presence of the TMD, a time history analysis has been performed considering a harmonic excitation at the tower-rotor resonance frequency equal to 0.6 Hz applied at the top node of the g introduction of a TMD, designed to operate at the tower-rotor resonance frequency, is useful to reduce the top displacement within the operation tolerance range. In fact, referring to the top displacement which leads the stop of the wind turbine, as shown in Fig. 8, the presence of the TMD leads to a non-negligible reduction of the top displacement: up to 43%. Fig. 8 shows the time history of the top displacement for the first 25 s of the analysis in both the configurations of the tower (with or without the TMD). It should be remarked that, after the first 6 s, the configuration without the TMD reaches the operational limit stop condition.

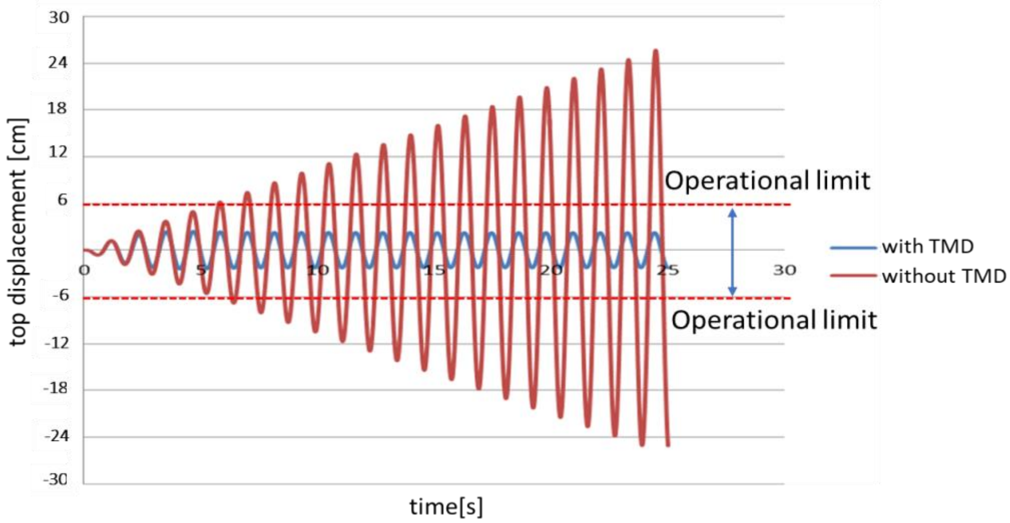

Figure 8. Comparison of the trend of the top displacement evaluated for the current tower with or without the TMD.

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From a practical point of view, an important aspect of this solution is that the installation of the TMD does not involve a significant stop of the wind turbine. The installation can be done directly in one working day by installing the TMD directly inside the tower (Fig. 9). The shape and the position of the TMD must be chosen in order to do not interfere with the internal stairs and lift located inside the tower for maintenance and inspection activities. The cost of this first solution ranges approximately between 30 and 50 k€.

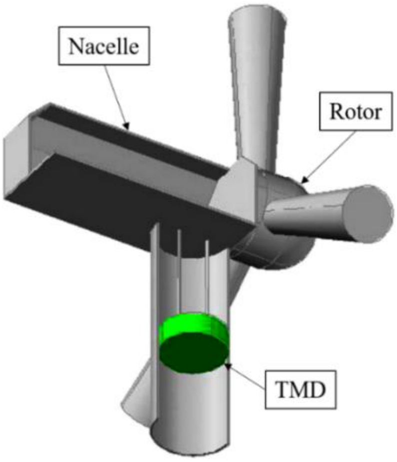

Figure 9. Example of TMD installation position.

As previously discussed, the second proposed solution concerns the modification of the tower height to bring it again to the initial design configuration, modifying the dynamic behaviour of the tower (Fig. 10). Also in this case, a finite element model was implemented with MIDAS Gen software where the tower is represented through tapered beam elements (as described in Table 2). The additional loads due to the presence of both rotor and generator and the values of the stiffnesses of the translational springs located at the base of the structure are the same described in the previous Section 3.

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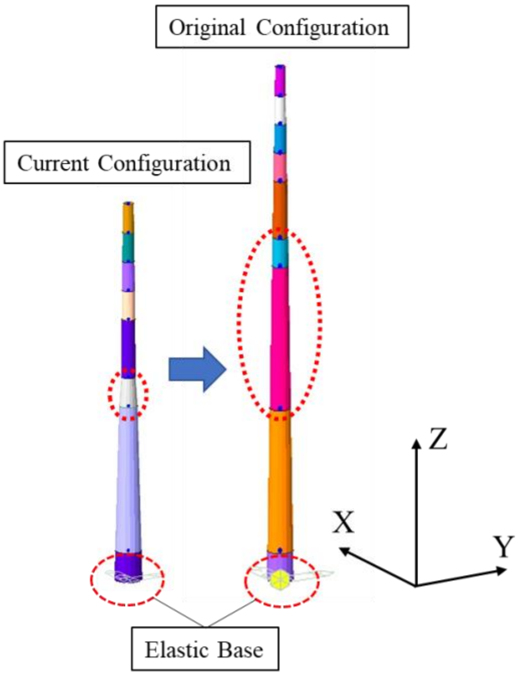

Figure 10. FEM of the "original" configuration.

Obviously, since the dynamic behaviour of the tower is comparable to the one of an inverse pendulum, by modifying the total length (and hence the stiffness of the structure) the frequency will change. In its "original" configuration, the tower is characterized by the following fundamental frequencies: 1.06 Hz for the first mode and 1.10 Hz for the second mode, both of them different from the operating frequencies of the rotor. Consequently, the resonance effects do not occur, and the maximum top displacement obtained during the time history analysis under harmonic forcing function is equal to 4.2 cm, lower than the limit displacement which leads to the wind turbine stop, indicated with the dashed red line in Fig. 8.

In this case, the maximum displacement is about 26% lower than the limit stop one. However, the choice of this intervention presents several critical aspects as the legal aspects deriving from the change in the height of the tower and the necessity of disassembly and assembly the whole tower. The estimated cost for this second solution is around 50 k€ considering just the realization of the new tubular steel segment plus transportation and assembling cost. In addition, considering the economic damages related to the complete stop of energy production during the downtime, the total cost can be estimated in about 70-80 k€.

5. CONCLUSIONS

In this paper, an experimental and numerical dynamic characterization of two steel wind turbines located in Italy is presented and discussed. Due to local Legislations, the height of the turbines has been modified from the original one: from 63m to 45.5 m. The intervention consisted of the removal of the longest tubular piece located in the centre of the tower and replaced with a shorter one. After their installation at the beginning of the operational activities, in normal wind conditions, both towers presented resonance problems. After a visual inspection aimed at understanding the health state of all the components of the wind turbines, nor damage in mechanical components nor cracks on structural components have been found. For this reason, an extensive modal identification experimental campaign has been then. This experimental activity is the starting point of the paper, in which the results obtained are discussed and reported. It has been clearly underlined that the first fundamental

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natural frequency of both towers in operational and non-operational conditions coincide exactly with the one of the rotors causing a resonance problem. As a consequence of resonance, huge displacements occurred on the top of the tower causing the lock of the system. In the current situation, these steel turbines can be used only with a low rotation of the rotor causing a very low energy production.

To solve the presented problems numerical finite element models (FEM) have been developed and calibrated, also discussing the influence of the soil-structure interaction (SSI) on the dynamic behaviour of these structures. In addition, two practical solutions to recover the functionality of the turbines, guaranteeing also the structural safety are proposed:

- the use of a TMD. This solution is already available on the market. A specific TMD could be installed directly inside the tower in a very short time. Some producers have already developed similar solution to the one proposed one, in Fig. 9, and the cost is quite limited if compared to the increase in energy production;
- the restoration of the initial tower. For example, if the Turbine 1 come back to 63m the first period became 1.10Hz instead of 0.60Hz. However, this solution is not feasible due to the high cost and high time of realization (the tower must be disassembly and assembly again).

In Table 8 a summary with the main peculiarities of the proposed solutions, comparing also the trend of the top displacement has been shown.

Table 8. Evaluation of the best solution to mitigate the resonance problem.

	Approximate cost	Time consuming	Feasibility	Top displacement reduction
TMD	30k-50k €	short operational stop	No problem	43%
Tower elongation	70k-80k €	long operational stop and transportation problems	Environmental laws can hamper this solution	23%

Since the re-use of wind turbines is a promising way toward sustainable energy production reducing the environmental impact of this kind of machines. Thus, the investigated problems can be common to the different producer of wind turbines from all over Europe the proposed solutions are of general validity and can be extended for many different cases.

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