POST-CRACKING BEHAVIOUR OF A WIND TURBINE CONCRETE TOWER

Javier Rodríguez, Joaquín Martí, Francisco Martínez

Principia, Spain

The paper deals with the dynamic performance of a simply reinforced concrete tower built using prefabricated elements. The main uncertainty of this strategy stems from the possible cracking of the concrete and its implications on the stiffness, natural frequency and dynamic amplification of the tower.

In 2006 an 80 m high prototype was built, supporting a 1.5 MW wind generator, carefully instrumented and test loaded to 80% of its design capacity. The prototype and installed instrumentation remained in operation for 3 years. Detailed calculations were carried out of the cracking induced in the concrete and its effects on the natural frequency of the tower, as a function of wind speed and orientation; the results were compared with the monitoring data.

It is concluded that numerical modelling with Abaqus allowed good predictions and interpretations of the observed response of the tower. Also, simply reinforced concrete is shown to be a good option for high towers; if the structure is well designed, the natural frequency will not migrate to a point where its proximity to the forcing frequency will lead to unacceptable levels of the dynamic amplification.

Keywords: wind turbine, concrete, cracking, natural frequency

1. Introduction

The last few years have witnessed a strong development in wind power generation, perhaps particularly in countries like Spain but also at a global scale. The general tendency has been towards greater tower heights, rotor diameters and rated powers. The majority of the existing wind turbines are supported on steel towers, which are shop fabricated in three segments, transported to site and assembled locally. However, beyond a certain size, this strategy must be abandoned because the dimensions of the tower segments no longer satisfy the limitations imposed by land transportation. Though in principle the tower could be decomposed in more than three parts, its reliability and durability can be expected to decrease as the number of connections increases.

A possible alternative is to use concrete instead of steel for building all or part of the tower. The concrete could be pre-stressed or simply reinforced, prefabricated or cast in situ. The use of simply reinforced concrete for this role may generate some concerns, on the grounds that its progressive cracking could reduce the tower stiffness, thereby bringing its resonant frequency closer to that of the excitation and hence increasing the dynamic amplification of the response.

The present paper deals with a reinforced concrete tower, built using prefabricated elements that are assembled in situ without any pre-stressing; the tower is 80 m high and supports an AW-77 wind turbine capable for 1.5 MW. A prototype was built, carefully instrumented and subjected to a load test in 2006 which imposed bending moments at the base equivalent to 80% of the design capacity, hence inducing extensive cracking in the tower. The data collected over three years has allowed conducting detailed investigations of the effects triggered by the large loads applied.

It is worth noting that this is not the first time that concrete is used for the tower supporting a wind turbine. For example, Enercon has developed 135 m towers supporting 6 MW turbines and GE Wind Energy has built 100 m towers in which the lower 70 m are made of concrete (Seidel, 2003). Also, design orientations for concrete towers have been published by Germanischer Lloyd Wind Energy (Klose, undated) and other institutions (Singh, 2007). However, they are concerned only with pre-stressed towers, reinforced concrete being used only in their foundations. It is in this sense that the present tower design can be considered innovative.

2. DESCRIPTION OF THE TOWER

As already mentioned, the tower is built using prefabricated reinforced concrete elements. In elevation the tower is divided into four segments, with diameters that decrease gradually from 6.3 m at the base to 2.7 m at the elevation of 78.2 m. Each vertical segment is made of 2 to 4 sectors, depending on the diameter at that elevation. The concrete used is C50/60 with a characteristic strength f_{ck} = 50 MPa in compression and $f_{ct,k}$ = 5.33 MPa in tension. The reinforcing bars are made of steel S500 with elastic limit f_{yk} = 500 MPa. In the present case the foundation footing rests on rock, a variable combination of sandstones and mudstones with high strength and stiffness. Figure 1 shows a photograph of the prototype.

Should the tower stiffness deteriorate because of concrete cracking, its natural frequency would of course be affected. The effect would be anisotropic, as it depends on the directions on which the cracks have been created and on whether the applied loads suffice to decompress the existing cracks. The concern for the frequency of the tower arises because, if it approaches that of the oscillatory loading, the dynamic amplification of the response will grow. In wind towers, the loads are applied at primarily two frequencies: P, which is that of rotation of the rotor, and 3P, that with which the blades pass in front of the tower.

In the AW-77 wind turbine the rotor adjusts its rotational speed as a function of the wind speed. As shown in Figure 3 the turbine does not operate for winds below 3 m/s; beyond that value a rotational speed of 11.6 rpm is maintained until the wind reaches 5 m/s. Beyond 9 m/s, the rotational speed is 16.7 rpm. And for wind velocities between 5 m/s and 9 m/s, the rotational speed grows linearly with the wind speed.

For low wind velocities, the rotor tries to develop the maximum torque; hence the orientation of the blades changes little and drag forces increase with the square of the wind speed. Once the peak power is reached, the orientation of the blades starts changing, since peak power can still be produced without increasing the drag forces. From a structural viewpoint, this is pictured in Figure 2, which shows the moments at the base of the tower as a function of the wind speed.

The maximum static loads contemplated in the design of the tower, quantified here through the bending moment developed at the base, were the following:

a) In operation, with winds below 25 m/s, a sustained load of 10 MNm and an instantaneous peak load of 20 MNm.

b) Not in operation, with winds above 25 m/s, a load of 40 MNm.

The tower may be subjected to a number of dynamic loads. Seismic loads do not govern the design of the tower in moderately seismic environments. Other dynamic loads, such as those associated with the wind variability and turbulence, are not the object of the present paper. Here the interest is focused on the loads applied with frequencies P and 3P, with P being a function of the current wind speed.

Figure 1. Prototype of the tower.

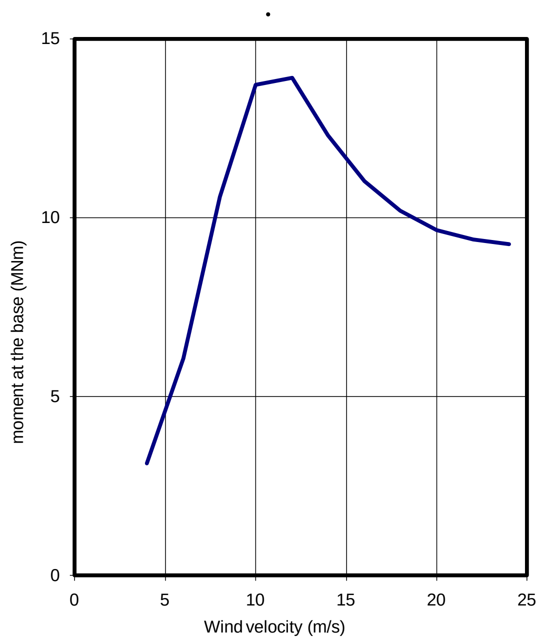


Figure 2. Moment at the base

3. LOAD TEST AND MONITORING DATA

In order to observe its effects on the behaviour of the tower, cracking was artificially induced by applying a high load at the top of the tower using a cable. The forces induced during the test, carried out on 6 October 2006, were first progressively increased to 550 kN, which corresponds to a bending moment of 30 MNm at the base; the load was then decreased to 0, followed by reloading to 367 kN and final unloading.

Figure 4 shows the loads exerted and the displacements caused at the top. The slope of the force-displacement relation evinces the reduction in stiffness experienced by the reinforced concrete sections with the progressive cracking induced.

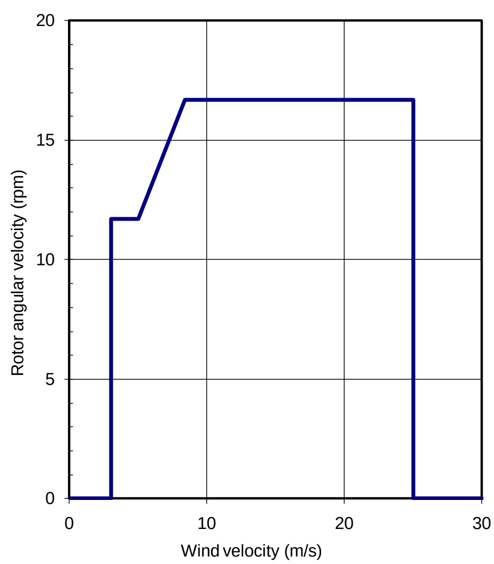


Figure 3. Rotational speed.

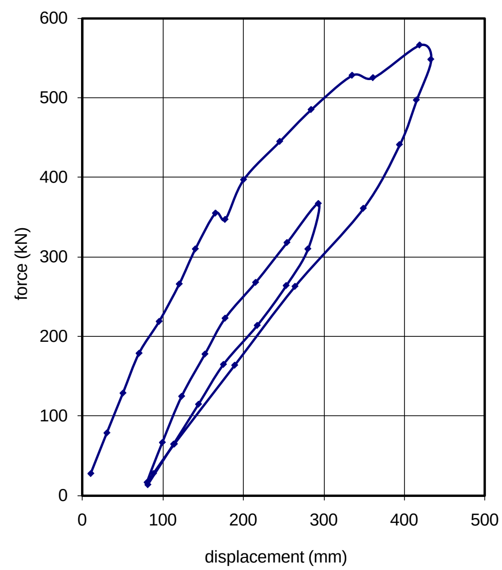
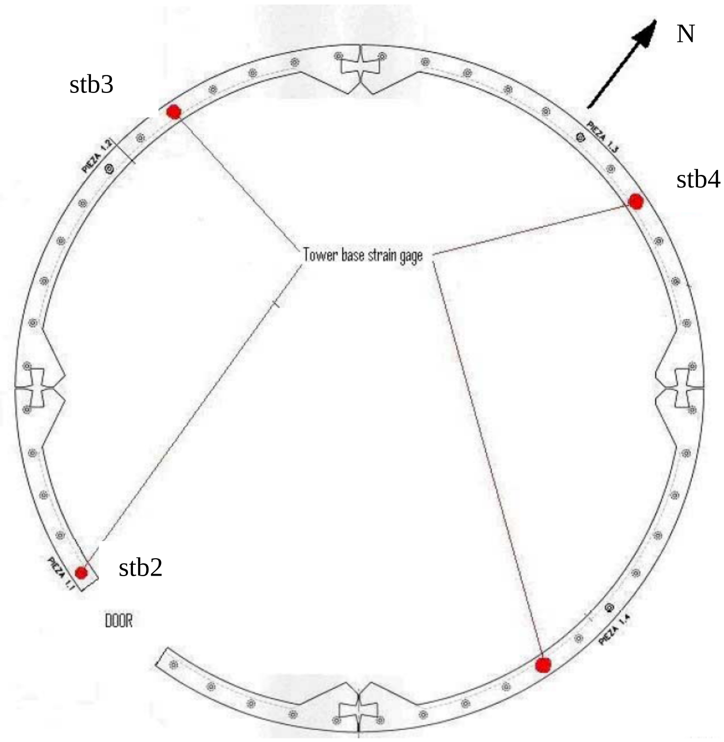


Figure 4. Force-displacement during load test.

Fourier analyses were conducted in all the available data sets in order to evaluate the natural frequency of the tower. Since the rotor speed is a function of the wind speed, the values of the frequencies P and 3P vary over the ranges indicated in Table 2. The spectra must reflect the frequency characteristics of both the excitation and the tower.

The tower was instrumented with a number of sensors, which recorded motions and forces at relevant points in the structure; Figure 5 shows the location of some of them. Other data being recorded included the wind speed and direction, as well as the rotational frequency of the rotor. The sampling frequency was 50 Hz, which allows a temporal resolution of 0.02 s.

The initial investigations were based on 25 data sets, each spanning 10 minutes, obtained at various times over the period from 11 August to 24 October 2006. Out of the 25 data sets, 11 precede the load test while the remaining 14 are more recent. Table 1 presents the dates and times of those records, together with the wind speed and orientation and the rotor speed during each of the 10-minute records. The horizontal division of the table corresponds to the timing of the load test.



Strain Gage	Orientation
stb2	191.25°
stb3	281.25°
stb4	11.25°

Figure 5. Strain gages location.

To analyse the behaviour of the tower, a finite element model (Figure 6) was constructed using Abaqus/Standard (SIMULIA, 2008). The first two natural modes of the uncracked tower appear in Figure 7.

Case	Wind velocity (m/s)	Wind direction (º)	Rotor velocity (rpm)
11.08.2006-04:30	19.0	13	16.7
14.08.2006-15:50	4.2	9	13.2
17.08.2006-02:00	4.6	280	13.0
17.08.2006-19:30	4.9	269	13.4
10.09.2006-18:40	3.9	195	11.6
11.09.2006-07:40	3.4	193	11.7
17.09.2006-15:30	5.2	9	12.0
22.09.2006-04:10	20.4	202	16.6
22.09.2006-04:20	20.0	195	16.7
03.10.2006-06:20	15.2	251	16.6
03.10.2006-06:30	14.7	256	16.6
07.10.2006-10:30	5.0	319	11.7
07.10.2006-12:20	5.3	318	11.7
07.10.2006-21:30	7.7	10	14.8
07.10.2006-21:40	7.9	12	15.0
08.10.2006-19:40	3.4	179	11.6
09.10.2006-13:20	5.4	190	11.8
11.10.2006-17:20	15.6	295	15.3
11.10.2006-17:50	17.5	311	16.6
12.10.2006-11:10	15.7	10	16.6
12.10.2006-17:20	16.7	16	16.7
17.10.2006-14:10	12.7	153	16.7
17.10.2006-15:30	12.9	153	16.6
24.10.2006-03:20	9.9	272	16.5
24.10.2006-03:30	10.0	266	16.6

Table 1. Data sets used in the study.

Wind velocity (m/s)	Rotor velocity (rpm)	Frequency P (Hz)	Frequency 3P (Hz)
3-5	11.6	0.19	0.58
5-9	11.6-16.7	0.19-0.28	0.58-0.83
9-25	16.7	0.28	0.83

Table 2. Frequency excitation as a function of wind velocity.

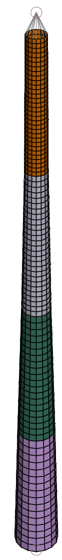


Figure 6. Model of the tower

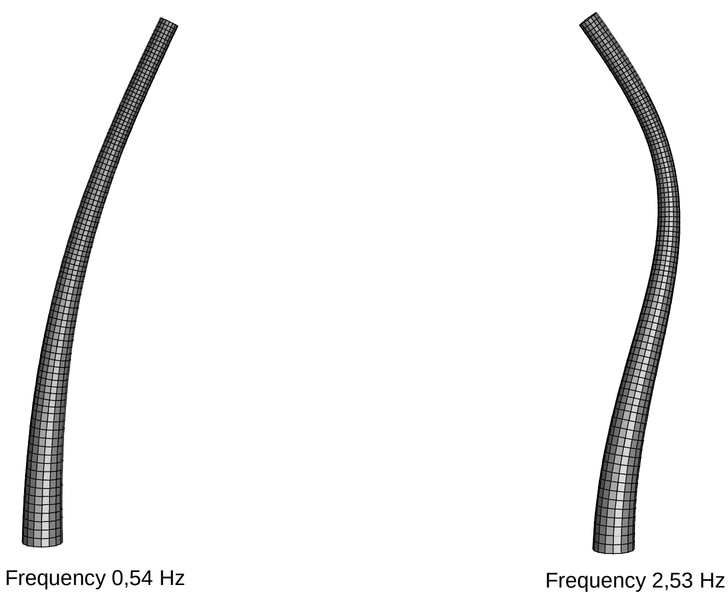


Figure 7. Two first modes of the intact tower.

The results of the Fourier analyses are summarised in Table 3, which shows the frequencies of the spectral peaks within the 0.4-0.6 Hz frequency range for each data set. The sensors used for preparing the table are the three located at the base and the three located at the first joint. The former are indicative of the maximum bending moments and the latter three allow confirming the specific vibration mode being excited.

Case	Veloc. (m/s)	Dir. (º)	Revol. (rpm)	Excit. freq. (Hz)		Inferred freq. (Hz)	
				P	3P	N-S	E-W
11.08.2006-04:30	19.0	13	16.7	0.28	0.83	0.52	0.50
14.08.2006-15:50	4.2	9	13.2	0.22	0.66	0.53	0.51
17.08.2006-02:00	4.6	280	13.0	0.2	0.65	-	0.53
17.08.2006-19:30	4.9	269	13.4	0.22	0.67	-	0.52
10.09.2006-18:40	3.9	195	11.6	0.19	0.58	-	-
11.09.2006-07:40	3.4	193	11.7	0.19	0.58	-	-
17.09.2006-15:30	5.2	9	12.0	0.20	0.60	-	0.52
22.09.2006-04:10	20.4	202	16.6	0.28	0.83	0.51	0.51
22.09.2006-04:20	20.0	195	16.7	0.28	0.83	0.51	0.51
03.10.2006-06:20	15.2	251	16.6	0.28	0.83	0.50	0.50
03.10.2006-06:30	14.7	256	16.6	0.28	0.83	0.51	0.50
07.10.2006-10:30	5.0	319	11.7	0.19	0.58	-	0.51
07.10.2006-12:20	5.3	318	11.7	0.19	0.58	0.51	-
07.10.2006-21:30	7.7	10	14.8	0.25	0.74	0.51	0.50
07.10.2006-21:40	7.9	12	15.0	0.25	0.75	0.51	0.49
08.10.2006-19:40	3.4	179	11.6	0.19	0.58	-	-
09.10.2006-13:20	5.4	190	11.8	0.20	0.59	-	0.51
11.10.2006-17:20	15.6	295	15.3	0.25	0.76	0.47	0.42
11.10.2006-17:50	17.5	311	16.6	0.28	0.83	0.47	0.43
12.10.2006-11:10	15.7	10	16.6	0.28	0.83	0.47	0.49
12.10.2006-17:20	16.7	16	16.7	0.28	0.83	0.47	0.47
17.10.2006-14:10	12.7	153	16.7	0.28	0.83	0.48	0.48
17.10.2006-15:30	12.9	153	16.6	0.28	0.83	0.49	0.49
24.10.2006-03:20	9.9	272	16.5	0.27	0.82	0.46	0.44
24.10.2006-03:30	10.0	266	17	0.28	0.83	0.48	0.46

Table 3. Detected frequencies.

The last two columns in the table list the vibration frequencies derived from sensors that are especially sensitive to N-S and E-W oscillations, respectively. No figures are quoted when the spectral peak at the forcing frequency 3P (on the order of 0.58 Hz for low wind speeds) and the natural frequency of the tower (about 0.52 Hz if uncracked or for low demand levels) are not sufficiently distinguishable to reach firm conclusions. The load test applied the maximum tensile load at 281º with respect to the north. The table shows that:

a) For low wind speeds, irrespective of wind direction, the natural frequency of the tower remains in the 0.50-0.52 Hz range because the wind loads are too low to decompress the existing cracks.

b) For high wind speeds, if the wind direction is far from 281º, some losses of stiffness can be observed: for example, the data sets from 12.10.2006 and 17.10.206 indicate frequencies of 0.47-0.49 Hz, which represent a decrease of 5-6%.

c) For high wind speeds and a wind direction near 281º, the static load applied by the wind opens the cracks produced in the test, and the oscillations with respect to this situation mobilise only the tangent stiffness. This is the case of the two data sets taken on 11.10.2006, which indicate frequencies of 0.42-0.43 Hz in the sensors that are especially sensitive to E-W oscillations, although the effect is barely perceptible in those that are primarily sensitive to N-S oscillations.

The last two data sets are characterised by a wind speed that is only intermediate, but with an unfavourable wind direction with respect to the existing cracks; the frequencies are then intermediate between the former cases, which can be taken as a confirmation of the previous interpretations. Furthermore, the lowest frequencies observed are consistent with the results of the load test; the line in Figure 8 represents the stiffness corresponding to a 0.42 Hz frequency, together with the load cycles produced in the test.

When simulating the load test, the concrete was characterised using the concrete model available in Abaqus/Standard, with plasticity and continuous damage (CDP). Plasticity, which can occur in both tension and compression, is combined with an isotropic damage model which uses one damage variable in tension and another one in compression. The constitutive model adopted seems to be the more appropriate one taking into account that the load test involves loading, unloading, reloading, unloading again, all followed by wind loading.

The calculations produced the load-displacement curve shown in Figure 9, which fits very closely the experimental observations. The same model allows determining the natural frequency of the tower under different loads. Such frequencies must be considered representative of the “intact” tower, i.e.: a tower that has not been loaded beyond the current level, hence the only cracks are those caused by the current loads. Figure 10 shows the progressive deterioration of the frequency as the applied loads increase, the latter still characterised by the moment at the base of the tower.

For obtaining the expected evolution of the natural frequency, an analysis was first carried out to determine the damage level caused at each material point in the concrete under the currently applied load. Then a damaged modulus was recalculated at each point by means of the user subroutine `UHYPEL`; this routine was specifically developed to determine damaged elastic moduli as a function of the computed values of the tensile plastic strains `PEEQT`. The recalculated modulus ensures unloading through the origin; otherwise the CDP model would have adopted a stiffer modulus, that would not have been representative of the concrete behavior during the oscillations being analysed.

As an additional verification, Figure 11 compares two relations between the virgin frequency and the base moment: one was produced by the model described above, while the other was generated using a beam model that includes the steel reinforcement and accounts for the inelastic behaviour of concrete.

4. IMPLICATIONS FOR OPERATION

4.1 Uncracked tower

Towers that have not been subjected to high wind loads, and hence remain uncracked, obviously maintain their original stiffness and frequency of oscillation. The first frequency is about 0.52 Hz and, being essentially a cantilever, that of the second mode is much higher. The dynamic amplification of the response depends on the proximity between the forcing and natural frequencies and also on the damping.

For an uncracked tower, the maximum amplification corresponds to the lower rotational speeds. When the wind blows with 3-5 m/s, the rotational speed is 11.6 rpm and the shadow of the blades crosses the tower at 0.58 Hz. In the absence of damping, the dynamic response to oscillatory loads at that frequency would be 4.1 times the static one

Damping depends on the size of the applied loads. If they are small, so is the damping; but this case is of little interest because it does not threaten the integrity of the tower. For large values of the loads, the expected damping values would be on the order of 5-7% of critical; in this situation the amplification factor would be 3.8.

4.2 Cracked tower

Once the applied loads are sufficiently high to reach bending moments of 30 MNm at the base of the tower, concrete will start to crack in several regions. Cracks do not affect the stiffness while they remain in compression; but if decompressed, any perturbations of the tower will only mobilise a smaller stiffness, an effect that will be shown by the vibration frequencies.

To decompress the existing cracks, the wind loads must be able to cancel the compression caused by the dead load. Also, if the cracking is not isotropic (as in the case of the load test described), the force must be suitably oriented to open the cracks.

These effects can be seen in Figure 12, which presents once again the frequencies for an intact tower, now together with those expected for a tower that was subjected to 30 MNm demands and is now being loaded in the same direction. The curve of the intact tower describes the frequency shift that would occur if the only cracks were those that the current loads are able to produce; the other curve represents the case in which the force only needs to decompress existing cracks, created by the prior application of 30 MNm demands.

When the orientation of the current force does not match that which produced the cracks, the effects would be intermediate between the two curves shown in the figure. Indeed, if the angle between them exceeds 90º, the pre-existing cracks would play no role since the applied force would not tend to decompress them.

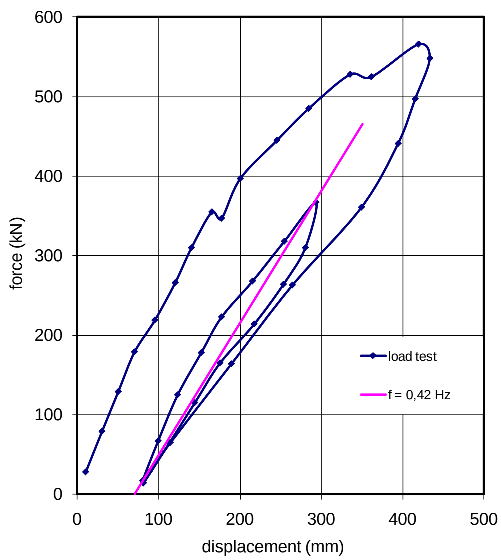


Figure 8. Equivalent stiffness for f = 0.42 Hz.

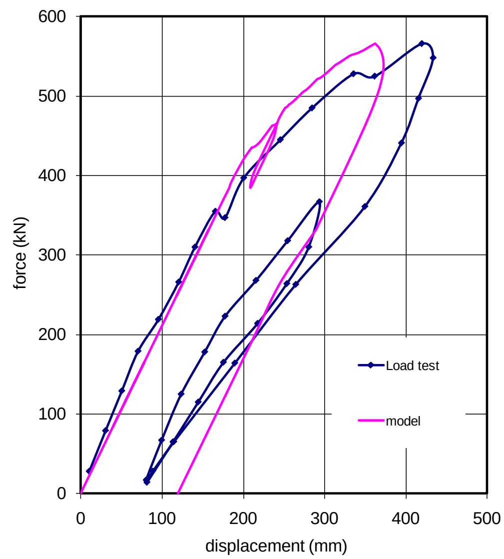


Figure 9. Calculation vs experiment.

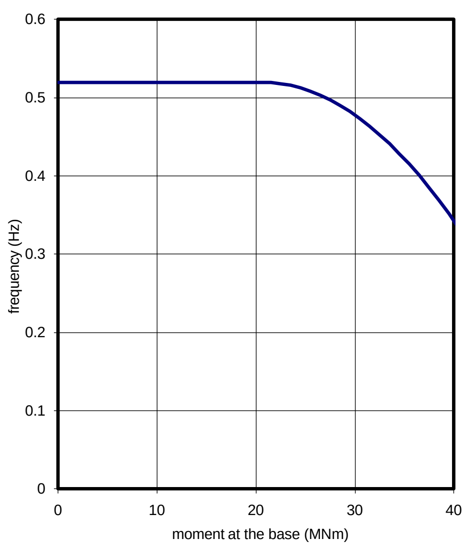


Figure 10. Frequency shift with increasing load in intact tower.

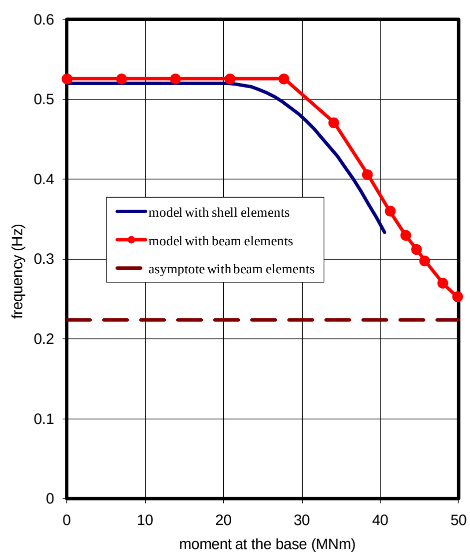


Figure 11. Comparison of results.

A moment of 30 MNm was used for preparing the previous figure because that was the load level imposed in the physical test. For other values of prior loading, the curves would be those presented in Figure 13. When the preexisting cracks cease to be compressed, the frequency starts to differ from that of an intact tower; that behaviour will be recovered when the applied loads exceed those that which produced the existing cracks.

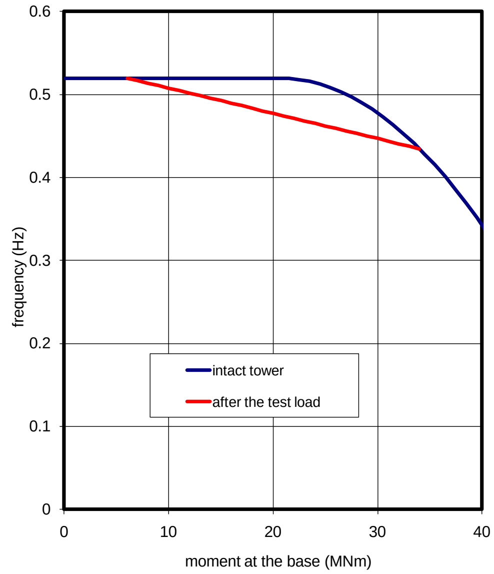


Figure 12. Frequency with and without previous cracking.

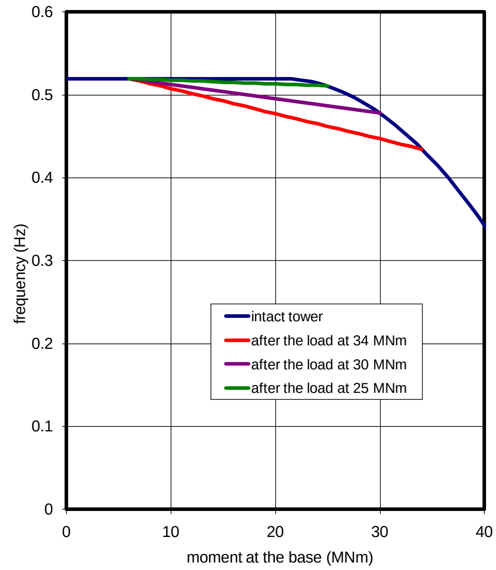


Figure 13. Frequencies with different levels of cracking.

As confirmation, Figure 14 shows the statistics of the frequency data taken over three years of monitoring. The sizes of the circles describe how often that particular frequency has been perceived in the response for each wind speed. In this regard, it is worth recalling that the moments at the base of the tower do not increase monotonically with the wind speed, but peak around 10-12 m/s. As can be seen, the situation remains stable and there is very good consistency between the theoretical expectations and the experimental measurements.

4.3 Response amplification

As already mentioned, it is the size of the demands that can give rise to problems in the tower, not the proximity of the forcing and natural frequencies. However, since the proximity of these two frequencies affects the dynamic amplification of the response to repetitive loads, it is important to consider the possible evolution of the frequency of the tower.

The dynamic amplification is plotted in Figure 15. The forcing frequency has been taken as 0.28 Hz, which is the frequency P for the higher wind speeds. The figure includes two curves, each for a different damping ratio. The more reasonable value of this parameter is 5%; lower values are obviously possible as well, but only for loads well below those of interest here. In any case, it is clear that the curves are not very different in the relevant frequency range.

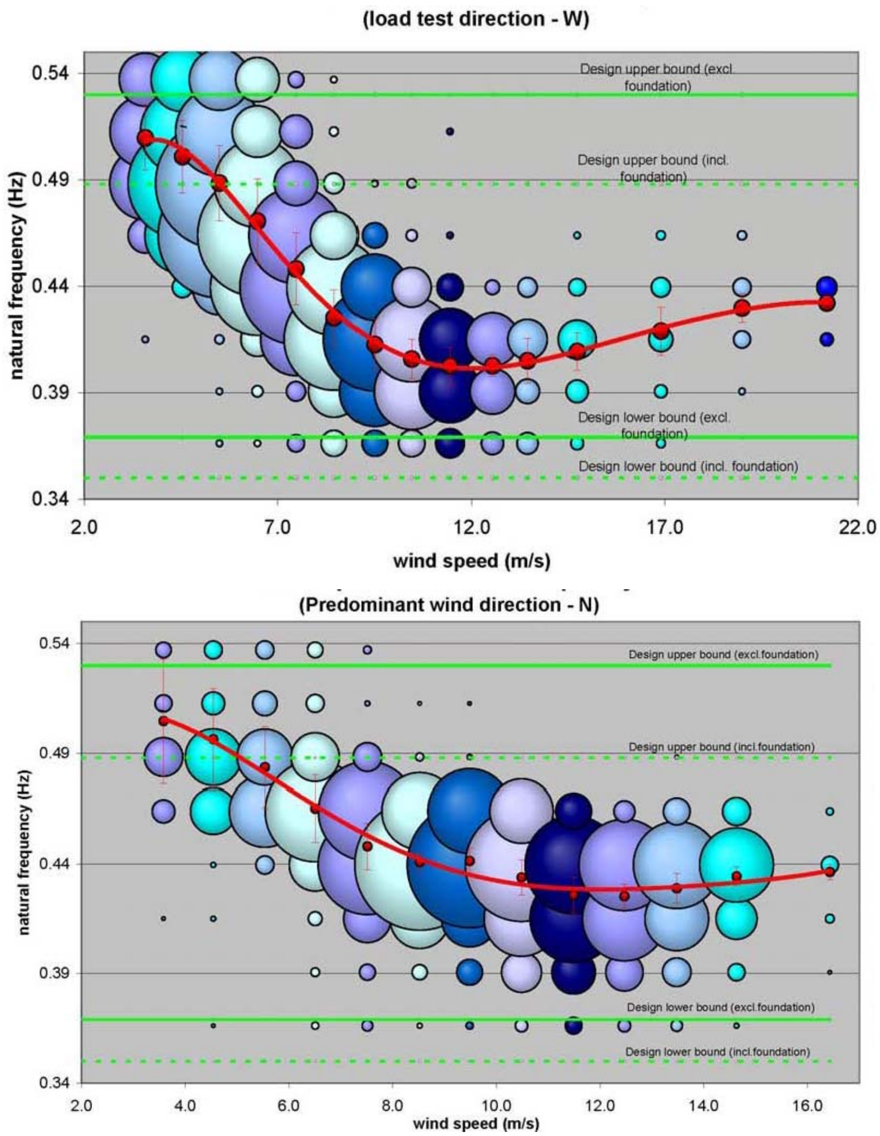


Figure 14. Frequencies observed for different wind directions and speeds.

With respect to the response to a static load, the response to cyclic loads applied with a frequency P (0.28 Hz) will be amplified by a factor of 1.41 because of its relative proximity to that of the intact tower (0.52 Hz). For a tower with the cracks generated by the loads that produce 30 MNm moments at the base, the dynamic amplification may grow to 1.80; this factor arises from the proximity of P and the frequency of a tower that already experienced 30 MNm at the base in the same direction (0.42 Hz). Such amplifications, undergone by the cyclic load applied with frequency P, are clearly of only very limited consequences.

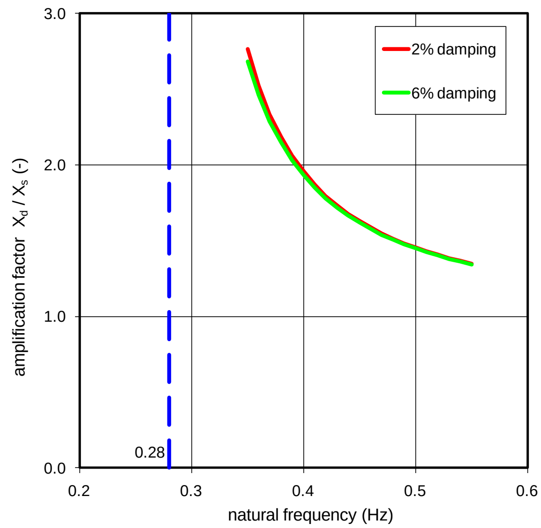


Figure 15. Amplification of the cyclic response.

5. CONCLUSIONS

The dynamic behaviour of a prefabricated, reinforced concrete tower has been investigated in some detail. The investigation was based on numerical modelling, together with the data produced by an instrumented, 80 m high prototype. The data included the dynamic records produced by the sensors during operation, as well as the information generated during a very severe load test to which the tower was subjected. The conclusions reached can be summarised as follows:

a) The existence of cracks may affect the frequency of the tower and hence its response to cyclic loads. However, if there are no cracks in the tensile region induced by the wind loads, or if they remain in compression under the applied loads, the effects will be barely perceptible. Numerical modelling has allowed a quantitative confirmation of these expectations.

b) The natural frequency of the prototype tower remained constant at 0.52 Hz until the load test was conducted and even afterwards while the applied loads are small or, in any case, if they are such that the existing cracks remain in compression. The largest effects appear when the applied loads are large and their orientation coincides with that of the load test, in which case the natural frequency may reduce to about 0.42-0.43 Hz.

c) The dynamic amplification of the response never increases beyond 1.8, even when the response frequency is as low as 0.42 Hz. As the maximum sustained load in operation is 10 MNm at the base and the tower is designed for 40 MNm, a 1.8 amplification does not generate any problems. Indeed this would remain so even if the amplification applied to the instantaneous peak load and not just to the sustained load.

d) In order to start affecting the natural frequency of the intact tower, the applied loads must develop moments at the base exceeding 20 MNm. At this level, which corresponds to the peak instantaneous load expected in operation, cracking of the concrete will start. Once cracked, the application of 6 MNm with the same orientation will suffice to decompress the existing cracks and start to affect the natural frequency.

6. REFERENCES

1. Klose, M., "Design of Concrete Structures for Offshore Wind Turbines", paper available from Risǿ National Laboratory, Wind Energy Department, Technical University of Denmark, undated.
2. Seidel, M., "Experiences with Two of the Word's Largest Wind Turbine Towers", 2003 EWEC European Wind Energy Conference and Exhibition, Madrid, 16-19 June, 2003.
3. Singh, A.N., "Concrete Construction for Wind Energy Towers", The Indian Concrete Journal, August, pp. 43-49, 2007.
4. SIMULIA, "Abaqus User's Manual, Version 6.8", Rhode Island, 2008.