Wind and Structures, *Vol. 23, No. 1 (2016) 75-90*
DOI: http://dx.doi.org/10.12989/was.2016.23.1.075

Reliability over time of wind turbines steel towers subjected to fatigue

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(Received November 6, 2015, Revised May 27, 2016, Accepted May 30, 2016)

Abstract. A probabilistic approach that combines structural demand hazard analysis with cumulative damage assessment is presented and applied to a steel tower of a wind turbine. The study presents the step by step procedure to compare the reliability over time of the structure subjected to fatigue, assuming: a) a binomial Weibull annual wind speed, and b) a traditional Weibull probability distribution function (PDF). The probabilistic analysis involves the calculation of force time simulated histories, fatigue analysis at the steel tower base, wind hazard curves and structural fragility curves. Differences in the structural reliability over time depending on the wind speed PDF assumed are found, and recommendations about selecting a real PDF are given.

Keywords: fatigue; structural reliability; binomial Weibull distribution; wind turbine tower; wind speed PDF

1. Introduction

It is usual, while doing feasibility studies in order to decide whether a site is adequate for installing a wind farm, to fit the annual variation of the hourly wind speed to a Weibull distribution. For most sites this probability distribution function (PDF) gives place to adequate results; however, there are sites which are exposed to different wind climates during different seasons, and it is well known that the Weibull PDF cannot represent all the wind regimes encountered in nature, as a consequence, the annual wind speed is best represented by other type of PDF (Carta *et al.* 2009) in order to minimize errors in the estimation of the energy produced by the wind energy conversion system.

A region in Mexico, known as “La Ventosa” in the state of Oaxaca, has an estimated potential of 2000MW of wind power. Jaramillo and Borja (2004) showed that a bimodal PDF must be used to compute the capacity factor for power plants installed in “La Ventosa”, otherwise the capacity factor might be underestimated 12% approximately. Such underestimation of the capacity factor can be translated in an underestimation of the forces acting on the turbine as well as of the fatigue damage suffered by the structure and, as a consequence, in the evaluation of the structural reliability over time.

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http://www.techno-press.org/?journal=was&subpage=8 ISSN: 1226-6116 (Print), 1598-6225 (Online)

Turbines were initially designed only for extreme loads that were expected during its 20-year life; however, after several structural failures it became evident that wind turbines are fatigue critical structures.

This study focuses in first place on the fatigue damage occurring at the base of the tower due to mode I fatigue failure (perpendicular to the crack). It is evaluated using a combination of time histories and wind data statistics. Once the fatigue structural damage at the base of the tower is estimated, a reliability analysis of the structure over time is performed, assuming that the annual speed is represented by a Weibull PDF (W.PDF), and alternatively, by a bimodal Weibull & Weibull (W&W) probability distribution. The consequences of using one or the other PDF are evaluated through the probability of failure over time of the structure.

The annual probability of failure is estimated here following the basic formulation proposed by Jalayer and Cornell (2003) for Performance-Based Seismic Engineering; but here it is extended to wind effects on the support structure of a wind turbine taking into account its fatigue structural deterioration over time. There are other extensions of the approach by Jalayer and Cornell (2003); for example, it has been extended to performance-based wind engineering (Ciampoli *et al*. 2011), to evaluate the seismic performance of buildings with corrosion deterioration (Celarec *et al.* 2011), to find the optimal time interval for inspection and maintenance of steel “jacket” platforms with fatigue deterioration (Torres and Ruiz 2007, Tolentino and Ruiz 2014), to obtain the confidence factor over time considering structural deterioration (Tolentino and Ruiz 2015), etc.

2. Reliability analysis approach

The approach proposed in this study to analyze the reliability changes over time of the structure consists of a fatigue analysis methodology coupled with a reliability analysis based on intensity measures. The general steps followed in the approach are:

1. Generate the turbulent wind field that is expected to act upon the rotor
2. Obtain the force time histories which are applied to the tower model
3. Perform a “step-by-step” time analysis of the tower to obtain the mechanical stresses at the base of the steel tower
4. Perform a fatigue analysis of the tower base using fracture mechanics theory
5. Apply a non-lineal static analysis (“pushover”) to the tower, and evaluate each structural damaged state of interest
6. Perform a reliability analysis of the tower, taking into account the wind hazard curve at the site where the tower is located.
7. Compare the probability of failure over time of the system assuming a W.PDF and, alternatively, a W&W distribution function.

In the following sections each of the concepts involved in the procedure is described, and an illustrative example of the method is presented.

2.1 Wind field

The IEC 61400-1 standard allows the use two different methods for turbulent wind field generation: 1) Veers method and 2) Mann method. Both produce reliable results; however, the first is used here because it is simpler and faster (due to its simplifications).

With regard to the wind field, the following hypothesis are made herein:

- Only the longitudinal component of the wind was considered; this simplification is considered adequate because in our case up to 90% of the wind energy is contained on the longitudinal direction.
- The atmosphere was assumed neutral, which is considered the most critical situation for designing wind turbines; however, it can lead to an overestimation of fatigue damage.
- The wind was considered acting in only one direction; for the site analyzed in this study this simplification seems valid as most winds follow a Northeast-Southwest direction (for other sites this hypothesis might not be valid).

Further information about the wind field generation method used on this work can be found in Veers (1988).

2.1.1 Weibull and W&W probability density functions

For fatigue calculations it is common to reduce the service life larger than 10^6 10-minute intervals to a few characteristic periods; to do this, the wind turbine operating range is divided into intervals and each interval is supposed to have a representative mean wind speed. The number of times each 10-minute interval with a certain average wind speed occurs in a year is commonly described by the Weibull PDF

$$f(u) = \frac{k}{c}\left(\frac{u}{c}\right)^{k-1} \exp\left[-\left(\frac{u}{c}\right)^{k}\right] \tag{1}$$

where u is the wind speed, c is a scale parameter and k is a form parameter, the relation between k and c is given by

$$\frac{\bar{U}}{c} = \Gamma\left(1 + \frac{1}{k}\right) \tag{2}$$

$$\frac{\sigma}{\bar{U}} = \frac{\sqrt{\Gamma(1+2/k) - \Gamma^2(1+1/k)}}{\Gamma(1+1/k)} \tag{3}$$

where $\bar{U}$ is the average wind speed, σ is the standard deviation of the wind speed and Γ the Gamma function. While Eq. (1) usually gives a good fit to wind data for wind energy applications, some specific locations present a bimodal probability distribution function (PDF), and the use of the unimodal Weibull distribution gives place to uncertain results. The Weibull & Weibull probability density function is given by

$$f(u) = p\left[\frac{k_1}{c_1}\left(\frac{u}{c_1}\right)^{k_1-1} \exp\left[-\left(\frac{u}{c_1}\right)^{k_1}\right]\right]$$
$$+(1-p)\left[\frac{k_2}{c_2}\left(\frac{u}{c_2}\right)^{k_2-1} \exp\left[-\left(\frac{u}{c_2}\right)^{k_2}\right]\right] \tag{4}$$

where p is weight component of the left side Weibull distribution, the rest of the parameters keep their definition from the common Weibull distribution (Eq. (1)). The relation between k and c is given by

$$\frac{\bar{U}_i}{c_i} = \Gamma\left(1 + \frac{1}{k_i}\right) \tag{5}$$

$$\frac{\sigma_i^2}{c_i^2} = \left[\Gamma\left(1 + \frac{2}{k_i}\right) - \Gamma^2\left(1 + \frac{1}{k_i}\right)\right] \tag{6}$$

2.2 Force time histories applied to the tower and mechanical stresses at the base of the structure

Following wind industry standard practices, the Blade Element Momentum (BEM) method is used here to generate the forces applied to the tower, and since we are not interested on the aeroelastic effects that usually occur on the blades, the forces are assumed to be concentrated at the hub position, which allows for further simplification of the analytical model. Glauert and blade tip losses corrections are made, following the theory presented by Moriarty and Hansen (2005).

The wind forces occurring on the support structure proper are obtained using the CFE Wind Design Manual (2008) recommendations for chimneys and similar structures, punctual forces were then applied in every node of the structure.

Once the forces are obtained it is possible to apply them to the Finite Element Method (FEM) model of the support tubular structure. The model was developed on the software SAP2000v17. The model characteristics can be found on the Case Study section of this work. Since SAP2000 is not an aeroelastic software, only the support structure was modeled. A "step-by-step" time analysis was developed for each mean wind speed of interest and the stress time histories at the tower base elements were calculated.

A statistical analysis of the stress time histories was necessary to obtain data to be used on the fatigue analysis of the elements. The main data needed were the mean stress, the number of stress cycles and the effective stress range corresponding to each of the time histories. The effective stress range allow us to represent the random amplitude stress time histories as an equivalent stress range that will cause the same crack growth for the same number of cycles. To obtain this data the Rainflow counting method suggested by the ASTM (2005) was applied, and the effective stress range was calculated using Miner's rule

$$R_{eff} = \left(\sum f_i {S_i}^3\right)^{\frac{1}{3}} \tag{7}$$

where f_i and S_i are the probability of occurrence and the stress range, respectively, for each interval in the stress histogram.

2.3 Fatigue analyis

Fatigue is considered an important failure mode for welded structures, and it is well known that wind turbines are fatigue critical machines (Do *et al.* 2014). There are several methodologies to analyze fatigue life of the structures; in this study fracture mechanics is used to account for the fatigue damage suffered over time by the steel tower. Fracture mechanics defines the local stress conditions around the crack in terms of the load, structure geometry and materials characteristics to determine the way the crack grows. The crack growth (da/dN) is defined here by the Paris-Erdogan (1963) equation

$$\frac{da}{dN} = C(\Delta K)^m \tag{8}$$

where C and m represent the material properties, and K is known as the stress intensity factor, which characterize the stress field around the crack.

Here the surface crack was assumed semi-elliptical (as shown in Fig. 1) and the equations for the stress intensity factor presented by Newman and Raju (1981) were used. Due to the characteristics of the model the expressions were simplified to consider an infinite length plate that is being affected only by tension, and to account only for the crack growth in depth (for an angle $\varphi = \pi/2$). The stress intensity factor range is given by

$$\Delta K_I = S_t F \sqrt{\pi\, a/Q} \tag{9}$$

$$Q = 1 + 1.464 \left(\frac{a}{r}\right)^{1.65} \tag{10}$$

where S_t is the remote tension stress range, a is the crack depth and Q is an elliptical crack form factor. The boundary effect correction factor F is given by (Newman and Raju 1981) where only the value at the outermost point of the crack is considered

$$F = \left[M_1 + M_2 \left(\frac{a}{t}\right)^2 + M_3 \left(\frac{a}{t}\right)^4\right] g \sqrt{\frac{a}{r}} \tag{11}$$

where

$$M_1 = 1.13 - 0.09 \left(\frac{a}{r}\right)$$

$$M_2 = -0.54 + \frac{0.89}{0.2 + (a/r)}$$

$$M_3 = 0.5 - \frac{1}{0.65 + (a/r)} + 14 \left(1 - \frac{a}{r}\right)^{24}$$

$$g = 1 + \left[0.1 + 0.35 \left(\frac{a}{t}\right)^2\right]$$

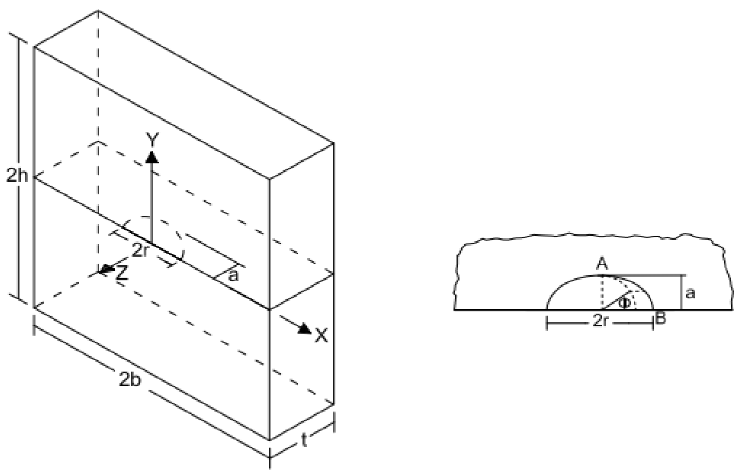

Fig. 1 Geometry of the proposed crack (Newman and Raju 1981)

Once all the crack growth parameters are known, it is possible to simulate the expected service life of the support structure and to obtain the fatigue damage over time at the points of interest; this is done by means of Monte Carlo simulation using the annual wind speed PDF corresponding to the site. The procedure used is as follows:

1. Obtain an average wind speed value for a 10-minute wind period from the annual average wind speed distribution.
2. For that value of average wind speed, obtain the mean stress value, the effective stress range and the number of cycles.
3. Using the Paris-Erdogan equation, estimate the crack growth suffered through the 10-minute period and save the current crack size.
4. Repeat steps 1-3 until the number of 10-minute periods of interest is reached.

2.4 Reliability analysis

The structural annual probability of failure P_F is obtained here using Eq. (12) (Jalayer and Cornell 2004, Montiel and Ruiz 2007). The structural capacity is represented by a fragility curve $F_R(y)$, and the distribution of the loading function is represented by the wind hazard curve corresponding to the site $v(y)$, both as functions of the intensity y.

$$P_F = \int_0^\infty \left|\frac{dv(y)}{dy}\right| F_R(y)dy \tag{12}$$

The annual probability of failure can then be expressed in terms of the reliability index β (Cornell 1969).

2.4.1 Calculation of the fragility curves

If the intensity measure is taken as the wind velocity u, then it is possible to express the fragility as a conditional probability to exceed a given limit state capacity U_c, as follows

$$F_R(u) = P[U_c \leq u] \tag{13}$$

If we assume that the distribution of the variable U_c is lognormal with mean λ and logarithmic standard deviation ξ, the fragility can be expressed in terms of the standard Gaussian distribution, as

$$F_R(u) = \Phi\left[\frac{\ln(u)-\lambda}{\xi}\right] \tag{14}$$

To express the capacity U_c in terms of wind speed we first need to define a limit state and its relation with a given wind speed. Here we get wind fragility curves based on the results of a static non lineal analysis ("pushover" analysis) (Lee and Rosowsky 2006). Once the "pushover" curve of the structure is known we can obtain the wind equivalent capacity curve correlating the base shear V with the wind loading, F_i, applied at n points of the structure

$$V = \sum_{i=1}^{n} F_i \tag{15}$$

Due to the variable nature of the wind load it is necessary to consider the base shear as a random variable with normal distribution (analogue as the wind speed distribution), so, using Monte Carlo simulation we obtain n wind equivalent capacity curves as shown in Fig. 2. It is noticed that this type of analysis is similar to the Incremental Dynamic Analysis proposed by Vamvatsikos and Cornell (2002); also it is noticed that for wind this procedure is acceptable only if it is considered as a monovariate stochastic process (Dimopoulos *et al.* 2015), as it was assumed in the present study.

Once a limit state is selected, it is possible to apply Eq. (14) to obtain the fragility curve associated with the limit state of interest (Fig. 3).

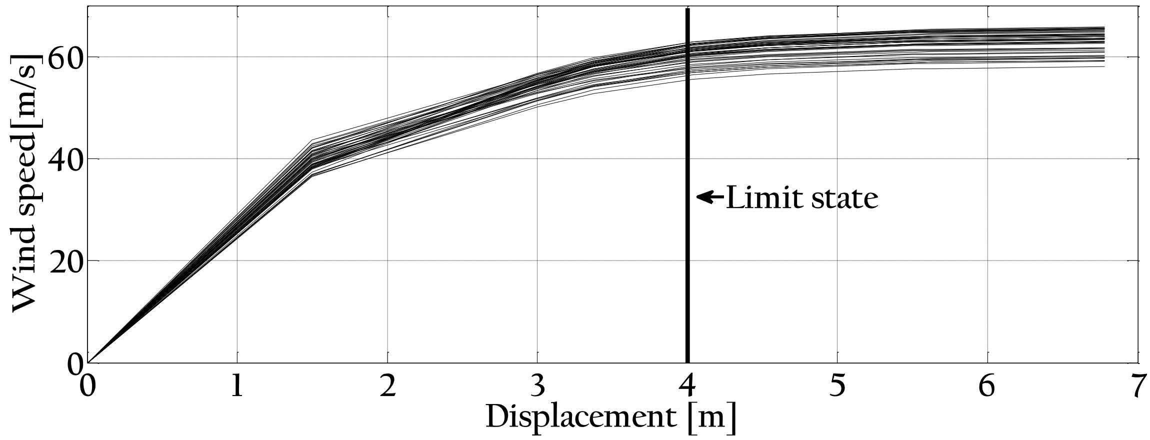

Fig. 2 Wind equivalent capacity curve

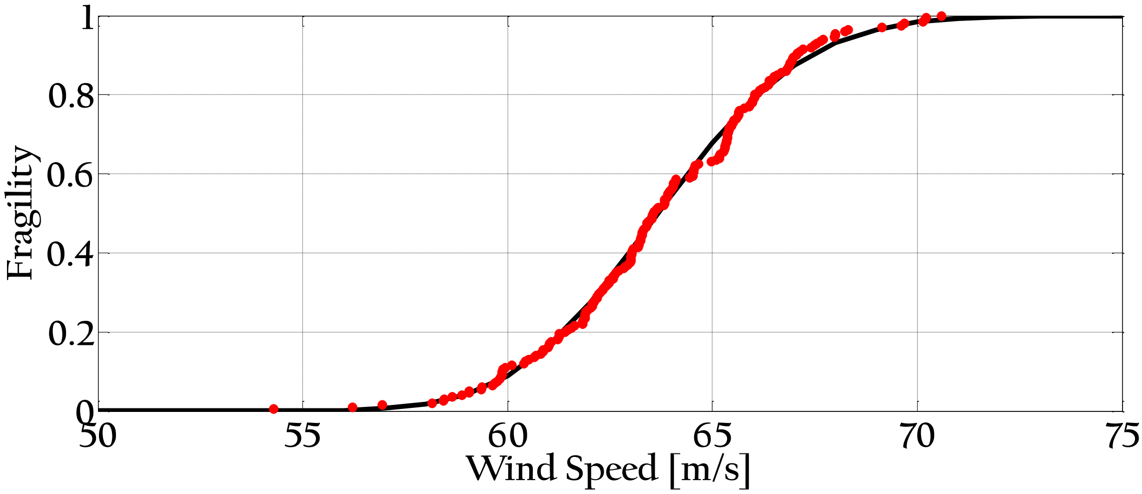

Fig. 3 Fragility curve for a wind speed intensity measure

2.4.2 Wind hazard

The wind hazard corresponding to a given intensity value is defined as the mean annual rate of a future event being higher than an intensity y. The wind speed annual maxima is adjusted to the generalized extreme value distribution (Gumbel distribution); the distribution mode μ and the scale factor b are determined with an expression that correlates the return period T with the wind speed maxima for that return interval u_T. These values can be obtained from wind hazard maps published by public institutions, which allows to build the wind speed hazard curve for the site, using the expression

$$u_T = \mu + b\left[-\ln\left(-\ln\left(1-\frac{1}{T}\right)\right)\right] \tag{16}$$

An example of the procedure mentioned at the beginning of Section 2 is presented in the following section.

3. Case study

A wind turbine located in “La Ventosa” area with coordinates $16°34.748'N, 94°48.981'W$ located in Oaxaca, Mexico, is analyzed. The area presents strong winds caused by pressure gradients between the atmosphere over the Gulf of Mexico and the warmer atmosphere over the Pacific Ocean, the wind caused by the pressure gradients flows toward the low pressure areas but it is blocked by the Cordillera Mountains and channeled through “Chivela” Pass. The bimodal Weibull & Weibull distribution parameters for the site are presented in Table 1, and the Weibull distribution parameters are shown in Table 2. A comparison of both distributions is shown in Fig. 4.

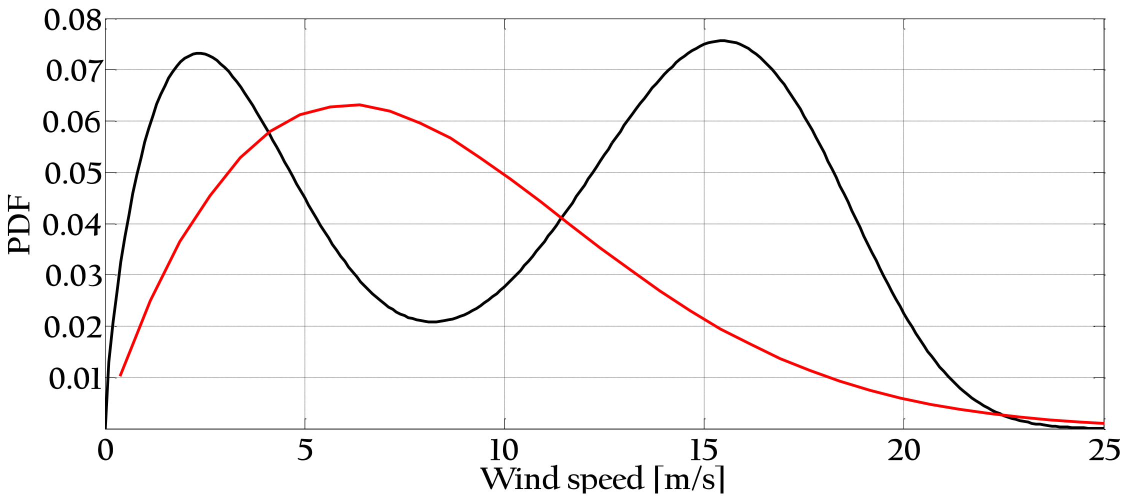

Fig. 4 Comparison of the annual wind speed distributions used

Table 1 Weibull & Weibull distribution parameters (Jaramillo and Borja 2004)

p	0.3799	[-]
$\overline{U}_1$	3.603	[m/s]
σ_1	2.212	[m/s]
k_1	1.674	[-]
c_1	4.034	[m/s]
$\overline{U}_2$	14.818	[m/s]
σ_2	3.256	[m/s]
k_2	5.232	[-]
c_2	16.097	[m/s]

Table 2 Weibull distribution parameters (Jaramillo and Borja, 2004)

$\overline{U}$	10.557	[m/s]
σ	6.169	[m/s]
k	1.768	[m/s]
c	11.861	[m/s]

The wind turbine analyzed is representative of the current and future Mexican wind parks. It is based on available information going from several manufacturers' public information to the models used by the academy, like the NREL 5MW wind turbine by Jonkman *et al*. (2009), and the DOWEC 6MW wind turbine by Lindenburg *et al*. (2003).

The steel tower is divided in 3 sections, with a lineal variation on diameter in height and a constant thickness for each section. The diameter goes from 4.3 m at the bottom to 2.13 m at the top of the tower. The thickness goes from 28 mm at the bottom section to 18 mm at the top. Steel specific weight is taken as 83.4 kN/m^3 to account the paint, screws, welds and joints that are not considered on the FEM model. The structural analysis was made using the commercial software SAP2000 which offers the necessary analysis capabilities required by the methodology proposed above. The base of the structure was considered fixed to account for the highly rigid foundations used in this type of structures. The main properties of the wind tower studied are presented on Table 3 and the FEM model used is shown in Fig. 5.

After the "step-by-step" analysis for each of the representative mean wind speed, the acting wind forces were generated and the fatigue damage calculations were carried using the Monte Carlo technique. For the fatigue analysis the annual average wind speed distribution, the mean stress distribution and the crack behavior are considered random variables, the rest of the data used was considered deterministic as there is not enough information that justifies any given probability distribution. The parameters used for the fatigue calculation are given in Table 4.

A "pushover" analysis was conducted using the software SAP 2000, and an analysis was made over time for each point of interest, for each analysis model the fatigue damage was considered as an equivalent thickness which decreases the total capacity of the support structure. Fig. 6 shows that for the undamaged state the tower can resist a total base shear of 1630 kN with a near collapse displacement of 5.08 m, both values are within the expected values that were obtained in previous studies such as the one presented by Kim *et al*.(2014). The “pushover” results corresponding to different time intervals are shown in Fig. 6. It can be observed that the ductility of the structure has a decrease as time passes; this is caused by an increment on base stresses as the crack growths and the equivalent thickness gets smaller, since the damage is not equally distributed on every member of the base, local fatigue failure can be reached before the collapsing load of the structure is attained.

Table 3 Tower main properties

Number of Blades	3	
Rated Power	2	[MW]
Blade Length	42.13	[m]
Rotor Diameter	84.26	[m]
Rotor Height	80	[m]
Rotor Weight	149	[kN]
Nacelle Weight	513	[kN]
Tower Weight	1496	[kN]
Blade Weight (ea)	58	[kN]
Steel grade	S355	
Yield stress	355	[MPa]
1st mode frequency	0.36702	[Hz]

Table 4 Fatigue calculation parameters

Variable	Mean	COV	Distribution	Reference
a_0	0.11	-	-	Moan (2000)
C	5.86 x 10^{-13}	-	-	BS7910 (2005)
m	2.88	-	-	BS7910 (2005)
a/c	0.5	-	-	Moan *et al*. (1993)
λ_{ac}	1	0.1	lognormal	Moan *et al*. (1993)
t	28	-	-	-
N	Variable with $\bar{U}$	-	-	-
R_{eff}	Variable with $\bar{U}$	-	-	-
$\bar{\sigma}$	Variable with $\bar{U}$	Variable with $\bar{U}$	Normal	-
$\bar{U}$	Tables 1 and 2	Tables 1 and 2	Tables 1 and 2	-

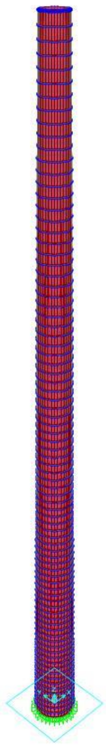

Fig. 5 Structural model used in SAP2000

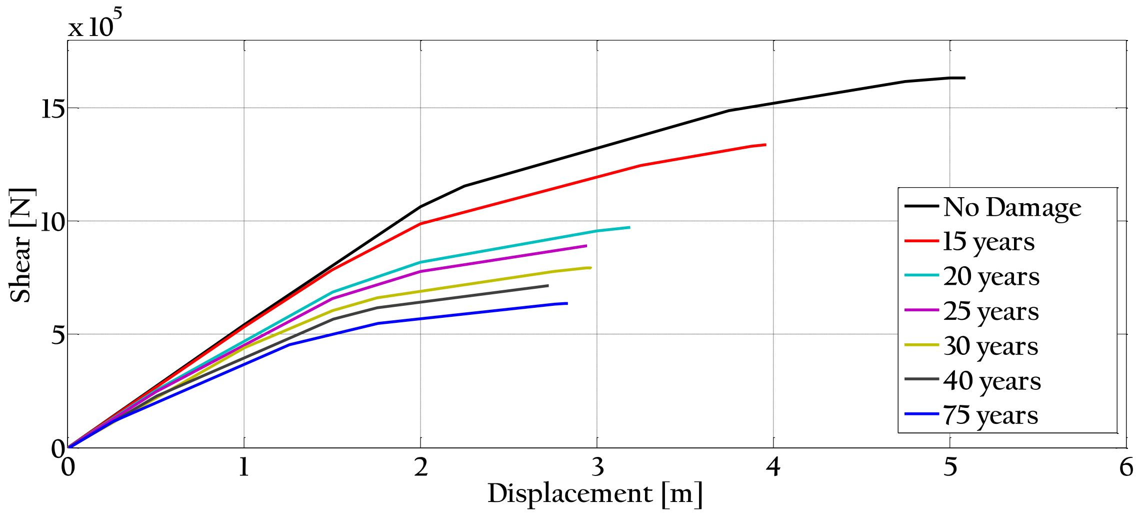

Fig. 6 Pushover results over time for a traditional Weibull distribution, a decrease of the capacity over time can be observed

Once the fatigue damage has been estimated, it is possible to obtain the fragility curve of the support structure over time, at the points of interest. Wind speed-base shear curves were randomly generated for eighteen values of mean wind speed considering that the base shear force follows a normal PDF. The wind turbine was considered parked for values exceeding common operational speeds. Finally, the wind fragility curves associated to different time intervals were developed following the procedure presented in Section 2.4.1. Fig. 7 shows the fragility curves over 10, 15, 25 and 75 years, corresponding to the traditional Weibull distribution, and Fig. 8 to the Weibull & Weibull distribution.

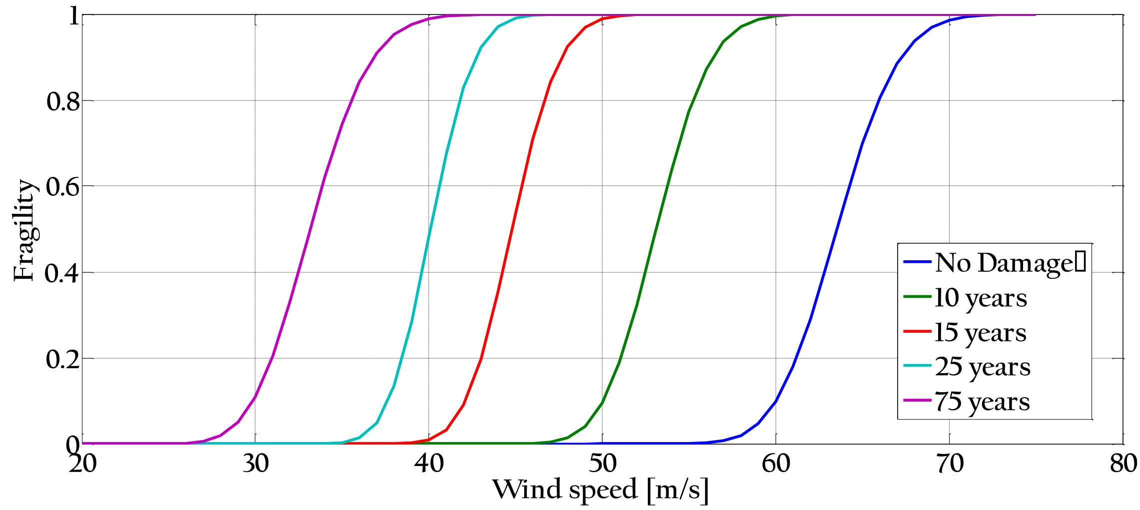

Fig. 7 Fragility curves for several points in time using a traditional Weibull distribution

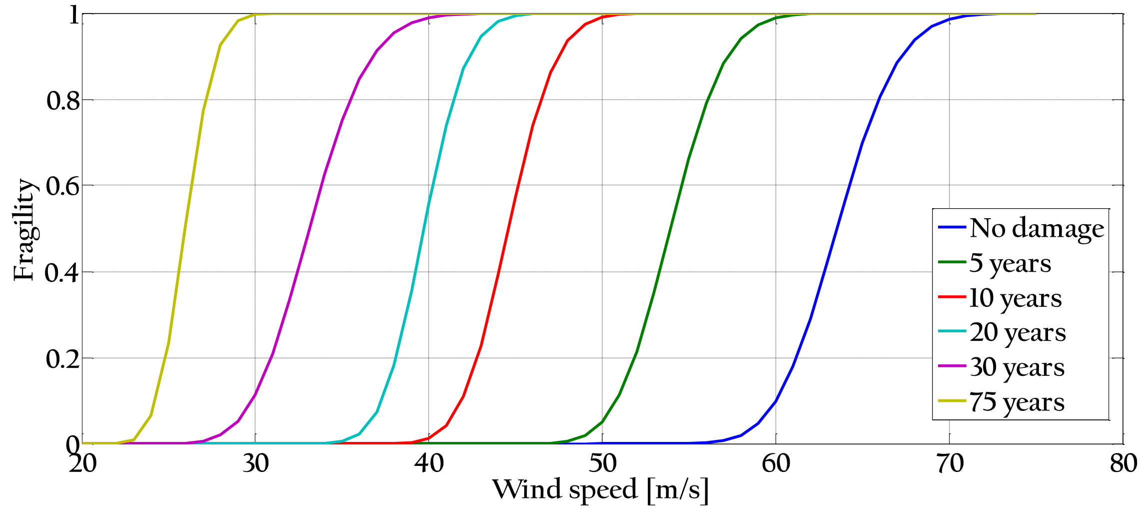

Fig. 8 Fragility curves for several points in time using a Weibull & Weibull distribution

Fig. 8 shows that the W&W distribution causes a faster damage accumulation compared with that associated with the traditional Weibull distribution (see Fig. 7).

The wind hazard curve was obtained using values of wind speed obtained from hazard maps provided by CFE (2008) for specific return intervals. The Gumbel distribution parameters were estimated using known values of wind speed for 2 different return periods. The data used to create the wind hazard curve is shown on Table 5. Linear interpolation was used to obtain the parameters at the site of study; however, it is possible that an underestimation of the wind hazard was made because the closest point of measurement is located at approximately 90 km of Salina Cruz, Oaxaca. The wind hazard curve used here is shown in Fig. 9.

Knowing the hazard curve and the fragility curves it is possible to estimate the reliability over time of the supporting structure using Eq. (12). Fig. 10 shows the changes of reliability over time for both mean annual wind speed distributions as well as the target reliability expected (indicated with a horizontal line). The target probability was calculated using the IEC 61-400 standard for a service life of 20 years, $P_F = 3.6 \times 10^{-3}$ and $\beta = 2.69$ as shown by Veldkamp (2007).

Table 5 Wind hazard curve data

u_{T10}	33.30	[m/s]
u_{T50}	37.85	[m/s]
u_{T200}	41.67	[m/s]
a	2.74	[-]
μ	27.17	[m/s]

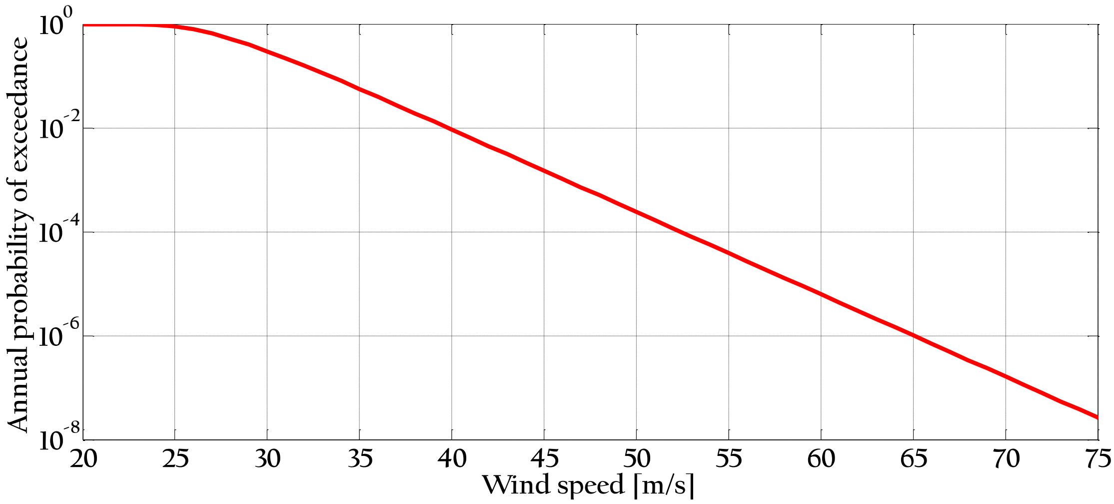

Fig. 9 Wind Hazard curve for “La Ventosa”, Oaxaca, Mexico

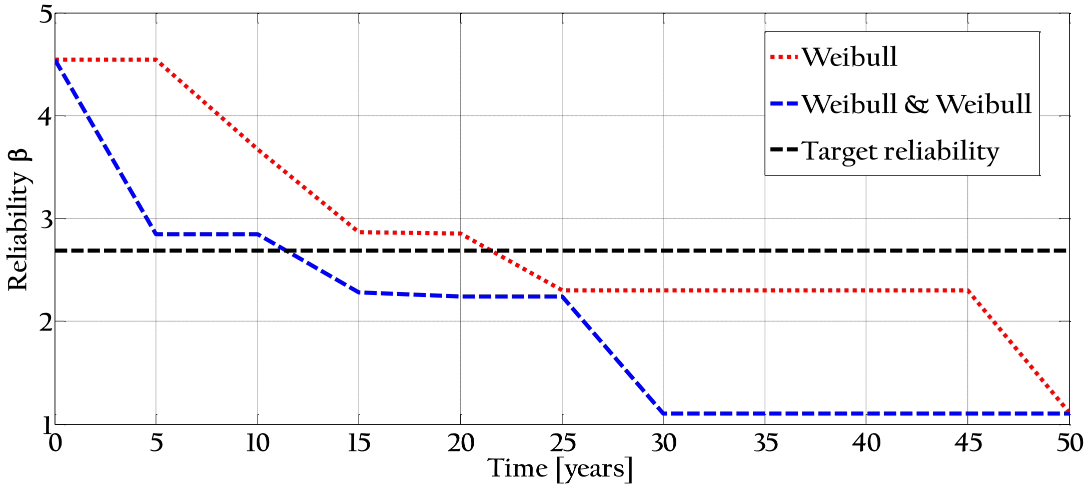

Fig. 10 Changes of the reliability index β over time, for both types of PDF

Fig. 10 shows that if a Weibull distribution is assumed for the annual wind speed, the time needed to reach the target reliability will be approximately 22 years, while if a Weilbull & Weilbull distribution is supposed (which is a more realistic assumption for the case analyzed) then, it will take approximately half of that time to reach the target (which is a smaller interval than the service life of 20 years). This example illustrates that the estimation of time needed to reach the target probability depends significantly on the PDF assumed in the calculations and that an erroneous assumption may lead to non-conservative estimation of the reliability over time of the structure.

4. Conclusions

An approach that combines structural demand hazard analysis with fatigue damage assessment applied to steel towers of wind turbines is presented and applied to a specific structure located in Oaxaca, Mexico.

It is shown that the probability distribution function (PDF) of the annual wind speed has a significant effect on the reliability over time of a wind turbine tower which presents structural degradation due to fatigue.

The illustrative example shows that if it is assumed that the annual wind speed follows a Weibull & Weibull PDF, instead of a traditional Weibull distribution function, a much faster fatigue damage rate is produced, because there is a higher probability that stresses that generate such damage occurs. As a consequence of assuming a bimodal distribution, the target tower reliability (according to the IEC 61-400 standard for a service life of 20 years) is reached during the first 12 years after the tower has been built.

It is recommended to adequately study the wind statistics data corresponding to the site of interest, and not using a "one fit all" distribution of the annual wind speed, ignoring that the real distribution could give place to unforeseen failures.

Acknowledgments

The first author thanks CONACYT for its support during his M. Eng studies. The authors wish to acknowledge DGAPA-UNAM for its support under the project PAPIIT-IN102114.

References

ASTM E1049 (1985), *Standard Practices for Cycle Counting in Fatigue Analysis*, West Conshohocken, USA.

BS 7910 (2005), *Guide to methods for assessing the acceptability of flaws in metallic structures*, London, UK.

Carta, J.A., Ramírez, P. and Velázquez, S. (2009), "A review of wind speed probability distributions used in wind energy analysis. Case studies in the Canary Islands", *Renewable and Sustainable Energy Reviews*, 13, 933-955.

Celarec, D., Vamvatsikos, D. and Dolsek, M. (2011), "Simplified estimation of seismic risk for reinforced concrete buildings with consideration of corrosion", *Bull. Earthq. Eng.*, **9**, 1137-1155.

CFE (2008), *Manual de diseño de obras civiles: Diseño por viento*, Mexico City, Mexico.

Ciampoli, M., Petrini, F. and Augusti, G. (2011), "Performance-based wind engineering: towards a general procedure", *Struct. Saf.*, **33**, 367-378.

Cornell A.C. (1969), "A Probability-Based structural code", *ACI J.*, No. 66-85.

Dimopoulous, C.A., Koulatsou, K., Petrini, F. and Gantes, C.J. (2015), "Assessment of stiffening type of the cutout in tubular wind turbine towers under artificial dynamic wind actions", *J. Comput. Nonlinear Dynam.*, **10**(4), 041004-041004-9.

Do, T.Q., Van de Lindt J.W. and Mahmoud H.N. (2014), "Fatigue life model including crack propagation for wind turbine tower base connections", *Struct. Congress*, 1606–1615.

IEC 61400-1(2005), *Wind Turbines - Part 1: Design Requirements*, Geneva, Switzerland.

Jalayer, F. and Cornell, C.A. (2003), "A technical framework for probability-based demand and capacity factor design (DCFD) seismic formats", *Pacific Earthquake Engineering Center* , Berkeley, USA.

Jaramillo, O.A. and Borja M.A. (2004), "Wind speed analysis in La Ventosa, Mexico: A bimodal probability distribution case", *Renew.Energ.*, **29,** 1613-1630.

Jonkman, J., Butterfield, S., Musial, W. and Scott, G. (2009), "Definition of a 5-MW reference wind turbine for offshore system development", *National Renewable Energy Laboratory*, Golden, USA.

Lee, K.H. and Rosowsky, D.V. (2006), "Fragility curves for woodframe structures subjected to lateral wind loads", *Wind Struct.*, **9**(3)**,** 217-230.

Lindenburg, C., Winkelaar D. and Van Der Hooft, E.L. (2003), "DOWEC 6 MW Pre-Design: Aero-elastic modelling of the DOWEC 6 MW pre-design in PHATAS", *Energy Research Center of the Netherlands,* Petten, The Netherlands.

Moan, T., Hovde, G.O. and Jiao, G. (1993), "Fatigue reliability analysis of offshore structures considering the effect of inspection and repair", *Proceedings of the 6th International Conference on Structural Safety and Reliability,* Innsbruck, Austria, August.

Moan, T. and Song, R. (2000), "Implications of inspection updating on system failure reliability of offshore structures", *J. Offshore Mech.Arct.*, **122**, 173-180.

Montiel, M.A. and Ruiz, S.E. (2007), "Influence of structural capacity uncertainty on seismic reliability of buildings under narrow-band motions", *Earthq. Eng. Struct. Dynam.*, **36**, 1915-1934.

Moriarty, P.J. and Hansen, C. (2005), "AeroDyn Theory Manual", *National Renewable Energy Laboratory*, Golden, USA.

Newman, J.C. and Raju, I.S. (1981), "An empirical stress-intensity factor equation for the surface crack", *Eng. Fract. Mech.*, **15**(1-2)**,** 185-192.

Paris, P.C. and Erdogan, F. (1963), "A critical analysis of crack propagation laws", *J. Basic Eng.*, **85**, 528-534.

SAP2000v17 (2015), Structural and earthquake engineering software, Computers and Structures, Inc.

Tolentino, D. and Ruiz, S.E. (2014), "Influence of structural deterioration over time on the optimal time interval for inspection and maintenance of structures", *Eng. Struct.*, **61**, 22-30.

Tolentino, D. and Ruiz, S.E. (2015), "Time-dependent confidence factor for structures with cumulative damage", **31**(1), *Earthq. Spectra*, 1-21.

Torres, M.A and Ruiz, S.E. (2007), "Structural reliability evaluation considering capacity degradation over time", *Eng. Struct.*, **29**, 2183-2192.

Vamvatsikos, D. and Cornell, C.A. (2002), "Incremental dynamic analysis", *Earthq. Eng. Struct. Dynam.*, **31**(3), 491-514.

Veers, P.S. (1988), "Three-Dimensional Wind Simulation", *Sandia National Laboratories*, Albuquerque, USA.

Veldkamp, H.F. (2006), *Chances in Wind Energy: A probabilistic Approach to Wind Turbine Fatigue Design*, Ph.D. Dissertation, Delft University, Delft, The Netherlands.

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