NUMERICAL MODELLING OF WELDED CONNECTIONS OF WIND TURBINE TOWERS UNDER FATIGUE LOADING

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Abstract.*The global switch to renewable energy sources has led to a boost in wind turbine implementation along with the use of other alternative energy resources. The most common wind energy structure is the tubular steel wind turbine tower, which is manufactured in 20-30m long tapered sections of circular shape that are mounted on site. The conical tubular subparts are composed of rolled steel plates welded longitudinally and circumferentially. These welded connections have been proved to be prone to fatigue failure and in many cases of wind turbine towers failure is observed in the form of cracking on welded joints. Towards wind turbine tower fatigue analysis, the present work focuses on two different wind turbine towers that have the same heights and bear the same moving mechanical parts at the top, but they differ in the tower shell thicknesses along the height. They are compared numerically and analytically towards fatigue loading using both loads proposed in design codes and artificial time-histories produced by NREL software. Useful conclusions have been derived from the comparison of the two towers on the determination of the tower shell thicknesses and on the effect of fatigue loading towards the material amount used for their construction.*

1 INTRODUCTION

The increased global electricity demand along with the need for greenhouse gas elimination have constituted the use ofrenewable energy sources imperative nowadays. Sustainable energy sources, such as wind energy, have significant advantages and a wide field for investigation. Their importance will become more evident in the near future and their objective towards clean energy production will be valuable. To this end, the improvement and optimization of wind production facilities is of great interest. On the civil engineering part, wind turbine towers have potential of structural detailing optimization which can result in more efficient and durable structures introducing their wider application, leading to improvements in energy production methods and costs.

Horizontal axis steel wind turbine towers are the most common configuration of wind power converters implemented, taking advantage of the material properties and geometrical advantages that shell geometry offers. Tubular towers usually consist of circular shaped sections of about 20-30 meters long that are mounted on site by means of bolted flanges with the use of pretensioned bolts. Each conical subsection is a steel plate rolled into a slightly tapered circular shape and welded with a seam lengthwise to constitute a closed ring, plus a circumferential welding seam to connect to the next can of the tower's ring as it can be observed in Fig.1. These rings are 2-2,5m long and are manufactured and welded in the workshop in order to form the longer cans that are finally assembled on site.The steel shell towersare designed as truncated cones and in order to sustain the great bending moment that develops at their bottom along with material saving reasons, are designed with a gradually increasing diameter from top to bottom along with a decreasing shell thickness from bottom to top. The welding design and verification is an important part of the whole tower structural design. In order to meet the strict requirements of the fatigue design, all welds realized in wind turbine towers are designed as full penetration butt welds of high quality [1].Fatigue analysis has a significant impact on the detailing and quality determination of both welds and bolts. In structural analysis of wind turbine towers, the structure is simulated as a full sized tower and very rarely special modeling of welds is taken into account in macroscopic models. According to European Standards, the design resistance of a full penetration butt weld, when welding is performed according to applicability provisions, is considered equal to the design resistance of the weaker of the parts connected [2].

Wind turbine tower structural analysis and investigation has been the field of interest of various research groups: Lavassas et al. [3], Bazeos et al. [4], Dimopoulos and Gantes [5].

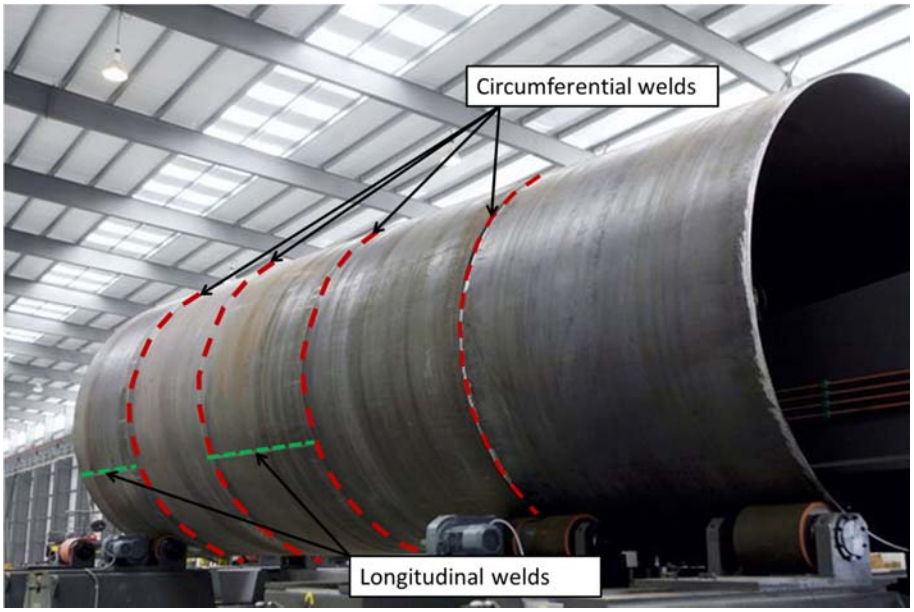

Figure 1.Position of longitudinal and circumferential welds (Source: http://www.gosanangelo.com)

Failures of wind turbine towers have been under investigation, assisting in the amelioration and optimization of certification and inspection procedures. Lee and Bang [6] performed a numerical analysis of a collapsed wind turbine tower, achieving accurate results. According to accident reports, 12% is attributed to structural failure and 6,8% is due to material fatigue [7]. There have been cases of wind turbine towers where failure is observed in the form of cracking on welded joints [8] and this type of failure is attributed to fatigue loading and inadequate design of the welds leading to concentration of stresses at those points. Special reduction factors are introduced for fatigue analysis of butt welds due to the great stress concentration observed [9]. As observed, fatigue is the main issue in the failures of welding in the connection of consequent tower parts, therefore alternative solutions are invented in order to replace the welding of the consequent tower parts, like the friction connection [10]. Steel towers and their fatigue life is estimated with the use of damage accumulation methods and the influence of several factors is further investigated [11]. The bottom flange weld of wind turbine towers is further investigated and optimized in the work of Jian et al. [12]. Unlike the bottom flange weld, circumferential welds of wind turbine towers are very rarely investigated against fatigue loading since industry often neglects their precise calculation and practices the increase of the tower shell thickness in order to avoid the fatigue check. In fatigue analysis of wind turbine towers, it is considered crucial to investigate all welds and bolts since details are more prone to fatigue. Recently there have been collapses of large wind turbine towers due to fatigue failure of circumferential welded joints [13] and sincelimited work has been devoted to the investigation of circumferential butt welds connecting consequent tower parts the present work focuses on the investigation of the fatigue life of wind turbine towers depending on the assessment of circumferential welds.

In the present research work, fatigue assessment of circumferential welds in wind turbine towers is addressed. Two identical towers in terms of height and bearing mechanical parts are taken into account, and differences in tower shell thicknesses are examined against fatigue loading. For the purposes of the herein presented investigation finite element analysis of the aforementioned towers is conducted with the use of the commercial software ABAQUS [14]. Instead of experimental wind data that are frequently used for the fatigue assessment, artificial wind time-histories produced by NREL [15] and NTWC [16] software are used for the present investigation. The loading time histories are produced for different wind mean speeds and Rainflowis used as a cycle counting method in order to produce the stress spectra. The life time of the two towers is compared using fatigue assessment based on the damage accumulation method and more specifically the Palmgren-Miner rule. In addition to the above mentioned method, the proposed procedure in Eurocode[17] for the assessment of dynamic loading of structures is also implemented and comparative results are assessed. Useful conclusions have been derived from the comparison of the two towers and the different methods used, on the determination of the tower shell thicknesses and on the effect of fatigue loadingtowards the material amount used for the construction of the towers. Fatigue loading is considered very important in the construction of structures under dynamic and cyclic loading and often has to be considered as the driving criterion in wind turbine tower design.

2 FATIGUE ANALYSIS

2.1 Definition and phenomenon

The phenomenon when a material is repeatedly bent back and forth developing a crack at the region of stress concentration, is called fatigue. This is observed even when the stress amplitude is within the elastic range of the material and the results of the phenomenon can be disastrous with structures collapsing without prior notice.When performing fatigue analysis the material properties and the dynamic loading cycles are very important. In shell structures like wind turbine towers, the mother material very rarely suffers from fatigue failure, whereas details like welds and bolts are more vulnerable to developing failures related to fatigue loading. Local connections, sudden decreases in dimensions, sharp edges, holes, welds etc. play an important role in the fatigue life of the global structure and they have to be addressed with special attention. In wind turbine towers, the stresses are repeatedly applied and relaxed by the resonant motion of the structure due to wind loading. Due to this cyclic loading steel hardening often occurs and becomes brittle, often leading to cracks and sudden collapses of the structures themselves without prior large deformation occurrence.

2.2 Methods incorporated

According to Eurocode [18] one of the methods used for fatigue assessment is the damage accumulation method. In the present work two different methods for obtaining the fatigue loading of the structure are incorporated, the time-history method named as TH method and the method proposed by Eurocode for taking into account dynamic loading named as D method.

The preliminary analysis of the structure consists of a linear static analysis of the tower against the fatigue loading received from the nacelle manufacturer. More specifically the loading consists of a horizontal force and an overturning moment that derive from the wind turbine operation and are considered the most important loads for fatigue assessment. The combination of the loading specifies the tower areas that are more prone to fatigue and the point that belongs to a circumferential weld and has the highest normal stress concentration is chosen for the fatigue assessment. This analysis serves for two reasons first because it specifies the point of the stress concentration that TH method will be implemented and second because it specifies the stress difference at the point of investigation needed for the calculation of cycles in D method.

For the D method, after the calculation of the stress difference at the point of concern and taking into account the analytical equation (1) given in Eurocode [17] and presented below:

$$\frac{\Delta S}{S_k} = 0{,}7 \cdot (\log(N_g))^2 - 17{,}4 \cdot \log(N_g) + 100 \quad (1)$$

where:

N_g is the number of loads for dynamic response

S_k is the effect due to a 50 years return period wind action

all numbers of loads for variable percentages of S_k are calculated. For nominal stress spectra, the fatigue strength is calculated by the extended fatigue strength curves as follows:

$$N_i = \left(\frac{\Delta\sigma_c}{\Delta\sigma_i}\right)^m \cdot 2 \cdot 10^6 \text{ with m=3 for } N \leq 5 \cdot 10^6 \quad (2)$$

$$N_i = \left(\frac{\Delta\sigma_c}{\Delta\sigma_i}\right)^m \cdot 5 \cdot 10^6 \text{ with m=5 for } 5 \cdot 10^6 \leq N \leq 10^8 \quad (3)$$

Having defined a detail category (S-N curve) for the tower detail under investigation and therefore the value $\Delta\sigma_c$, the damage of the structure can be calculated by applying the linear damage accumulation method, usually called Palmgren-Miner rule. The cumulated damage is given by eqn (4):

$$D_d = \sum_i^n \frac{n_{Ei}}{N_i} \quad (4)$$

where:

n_{Ei} isthenumber of cycles associated with the stress range $\gamma_{Ff}\Delta\sigma_i$ for band i in the factored spectrum

N_i is the endurance (in cycles) obtained from the factored $\frac{\Delta\sigma_c}{\gamma_{Mf}} - N_R$ curve for a stress range of $\gamma_{Ff}\Delta\sigma_i$

The reciprocal value of the damage equals the approximated lifetime of the steel tower. In the case of this study n_{Ei} is the number of loads for dynamic response and the endurance is obtained from eqs (2) and (3).

For the TH method, the point with the highest stress concentration is chosen from the preliminary analysis for both towers under investigation.The primary data used in fatigue analysis are stress time-histories that come from the analysis of the structure under real loading or artificial loading. The rainflow counting method is mainly used to convert the complex stress time histories into simple cyclic loadings or stress range spectra. The result of the rainflow counting method can be transformed into a spectrum of amplitudes of stress cycles in one year. After the calculation of the amplitude spectrum, the linear damage accumulation method is used in order to calculate the fatigue life of the structure.

3 NUMERICAL MODELS

3.1 Towers description

The wind turbine towers under investigation share the same height of 76.15 meters and comply with certain restrictionaltransportation requirements concerning maximum top diameter and maximum tower part length. The maximum length of the tower sections is usually governed by requirements to allow for transportation while the upper limit for the outer diameter of land-based wind turbine towers is usually governed by restrictions by the maximum clearance under highway bridges. The maximum value for the top tower diameter is defined as 3.0 m and the maximum bottom diameter is defined as 5.0m while the maximum length of a transported element is 26m. Following the above mentioned restrictions, the towers under investigation consist of subsections of 2.2m height and only the top section is set to 2m in order to complete the full tower length. The top diameter is 3,0 meters and the bottom one is 4,3 meters. The shell thicknesses of each tower can be observed in Fig. 2. The first tower [A] complies with the regulations introduced by the manufacturer and has been designed using finite element analysis for extreme wind conditions. The second tower [B] is also designed under extreme wind conditions and additional criteria for shell thicknesses are investigated in order to minimize the tower mass in the scientific work conducted by Bzdawka [19].

3.2 Loadsdetermintation

As mentioned already, the fatigue loads taken into account in the preliminary analysis of the wind turbine towers were provided by the manufacturers and were the fatigue loads from the machinery. These data were the horizontal force of F=75,5kN and the horizontal axis moment of M=1091kNm. The tower weight was not taken into account in the fatigue analysis.

For the analyses of the TH method, the loads are calculated from NREL software and for time and data saving reasons only the two major loading histories on the top of the tower are taken into account. Theconcentratedloads are applied at the top of thetower to a referencepointtakingintoaccounttheeccentricity of the rotor position. Thepointwheretheloads are appliedispositioned at the top of thetowerhavingthe center of gravityshiftedhorizontally +0:725m fromthe axis of thetower and vertically +0:50m abovetheupperflangelevel (+76:15m). The time-histories applied at the top of thetower are for variable hubheightwindspeeds in order to cover all the spectrum of operational winds.

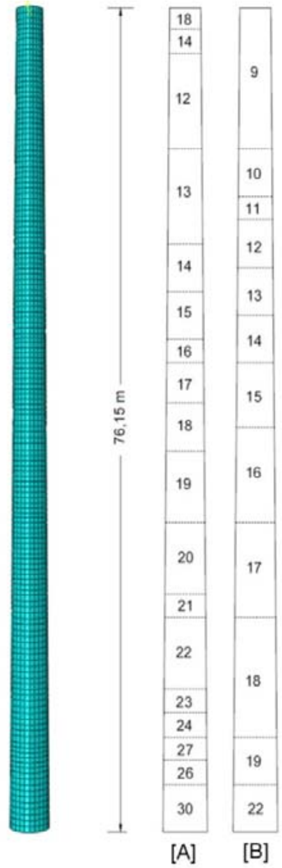

Figure 2.Shell thicknesses of investigated towers [A] and [B].

4 RESULTS

4.1 TH Method - Stress range histograms

At the hub height of the two towers under investigation, artificial time histories for shear and moment loading are implemented. These data are obtained from the free software; Turbsim, Fast and Aerodyn for all the wind speeds taken into consideration. After the numerical analysis of the two towers, the normal stresses $\sigma_{zz}$of the tower shell are obtained, since the tensile stresses are the ruling factor leading to fatigue failure. In Fig.3 and Fig. 4 the stress histories σ_{zz} of the middle surface of the towers [A] and [B] respectively are presented for the various wind speeds.

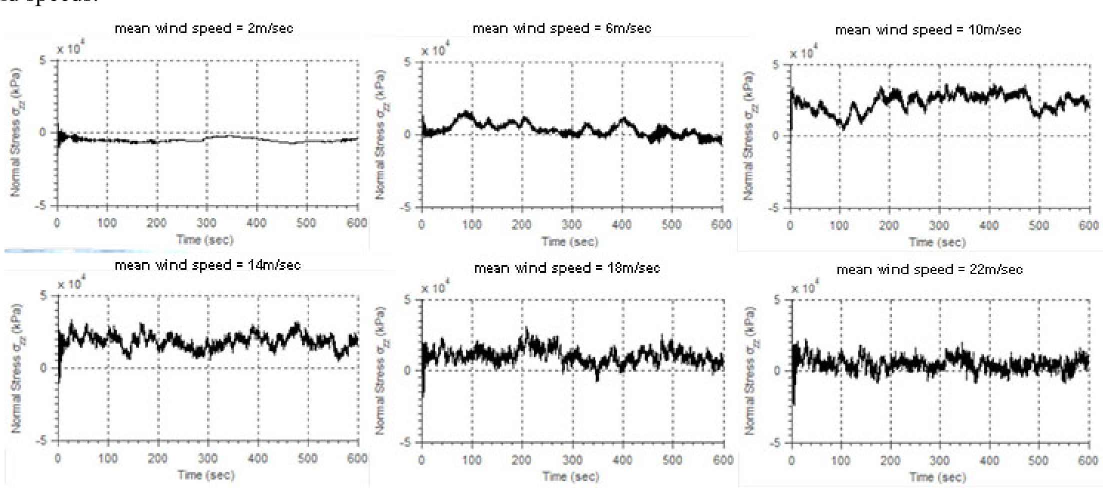

Figure 3. Tower [A] stress histories

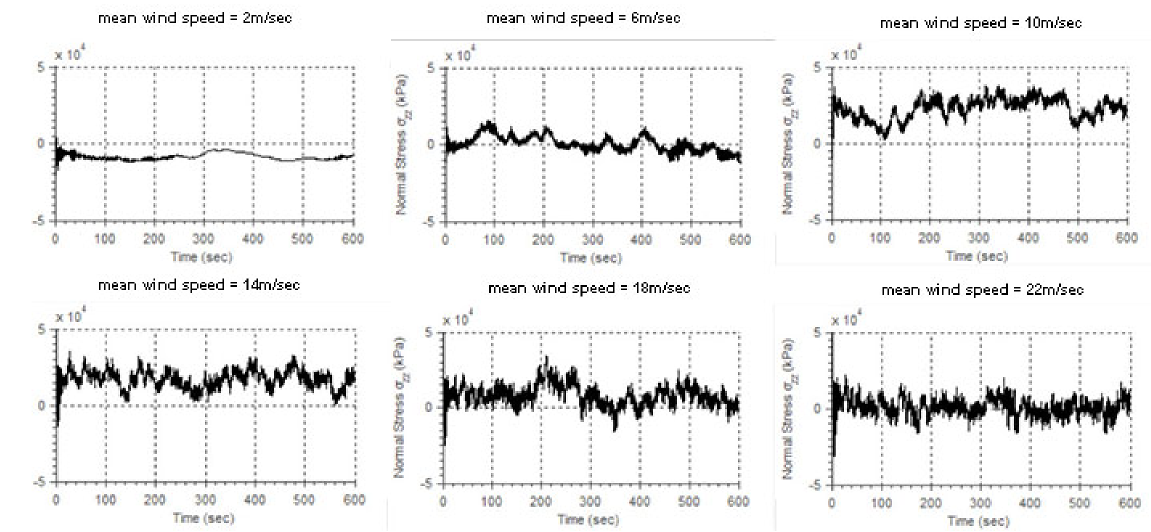

Figure 4. Tower [B] stress histories

These above presented stress histories are obtained for the point of the circumferential welding that is more vulnerable to fatigue failure.

4.2 TH Method - Cycle Counting

The stress diagrams are time related and great variability of frequencies and ranges are observed. The complex stress time histories are converted to histograms of number of cycles and stress levels. This is realized in practice by converting the simple time history to simple cyclic loading by picking the peaks for tensile and the valleys for compressive stresses through the reservoir or the rainflow method. In the present investigation the rainflow cycle counting method is selected and it is applied intomatlab code. For the area of the towers that is defined by the preliminary analysis, the stress level histograms are produced for each one of the six wind speed time histories as shown in Fig.5 and Fig.6. Since fatigue check is referring to normal operating conditions, the first 10 seconds of the time histories are neglected due to the presence of signal noise deriving from the launching of the machinery.

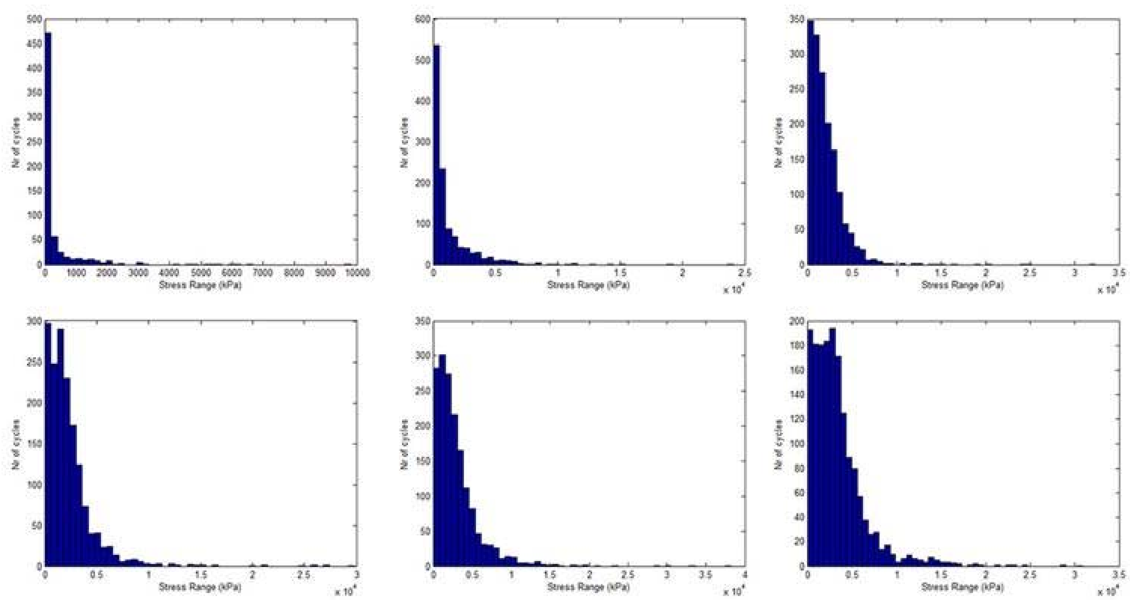

Figure 5. Tower [A] stress range histograms for variable mean wind speeds

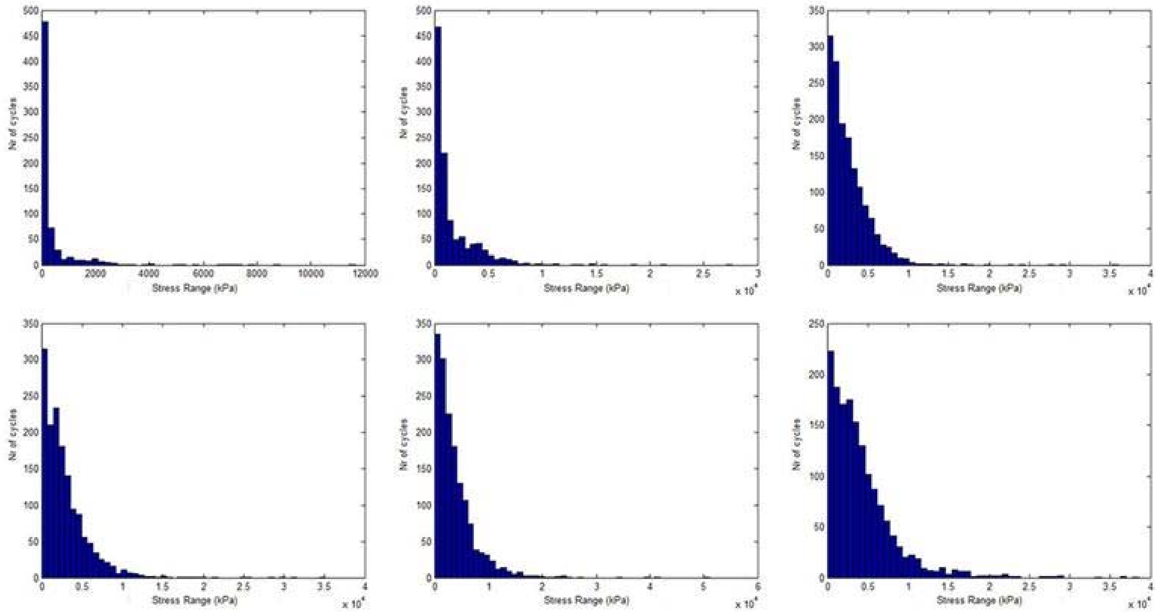

Figure 6. Tower [B] stress range histograms for variable mean wind speeds

4.3 TH Method – Damage accumulation calculation

The above mentioned stress range histograms present the annual number of cycles for each stress level based on the number of cycles in the 10 minute wind. According to IEC 61400-1 [20] the distribution of wind speeds over an extended period of time is given by the Rayleigh or the Weibull distribution.The Rayleigh distribution is used in the present investigation and the probability density function and cumulative density function are presented in Fig.6. According to the same standard the cumulative probability function is given by Equation (2).

$$P_R(V_{hub}) = 1 - e^{-\pi\left(\frac{V_{hub}}{2V_{ave}}\right)^2} \tag{2}$$

where, $V_{ave} = 0.2 * V_{ref}$ and for wind turbine class II [20] $V_{ref} = 42.5$ m/sec.

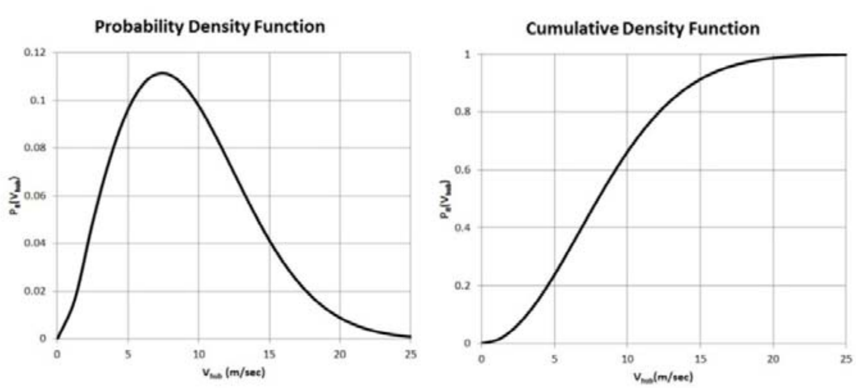

Figure 6.Rayleigh probability density function and cumulative density function

The annual number of cycles derives from the sum of the cycles of each 10 minute wind multiplied by the relevant probability of occurrence, multiplied by the number of 10 minute durations in one year. After these calculations, the annual stress range spectrum for each tower is calculated and presented in Fig.7 and Fig. 8.

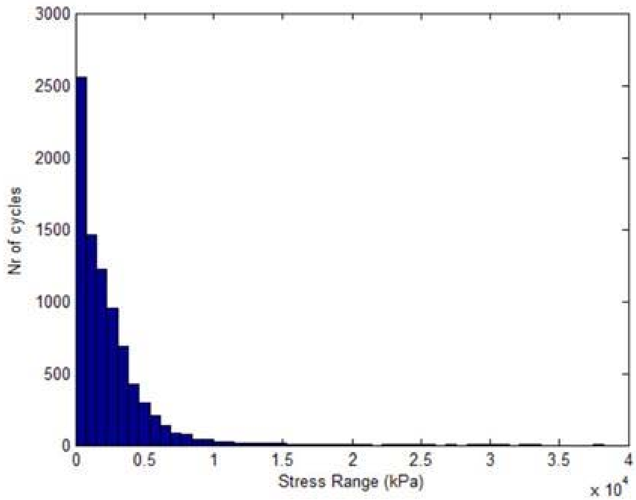

Figure 7. Tower [A] annual stress range spectrum

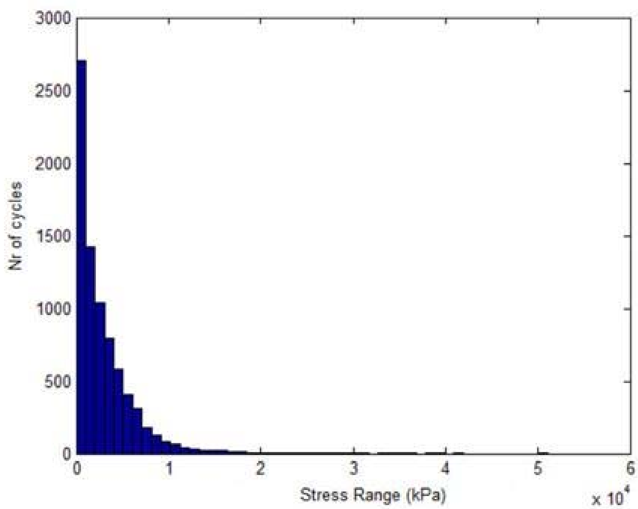

Figure 8. Tower [B] annual stress range spectrum

When structures are loaded with amplitude varying loads and with different number of cycles corresponding to each amplitude, cumulative damage is gathered at special structural details that are more vulnerable. The cumulative damage for the tower details under investigation is calculated following the Palmgren-Miner rule. The detail category that corresponds to the circumferential weld between consequent tower parts, according to EN 1993-1-9 [18] has fatigue stress capacity of $\Delta\sigma_c$=80 MPa. The factored capacity curve that corresponds to this structural detail is presented in Fig. 9.

For the fatigue check of the structural details of the two towers using the TH method, the cumulative damage has to comply with the criterion of Dd<1.0. The annual cumulative damage for tower [A] is 0.0007 and for tower [B] is 0.0019. The fatigue check criterion is fulfilled for both towers and the fatigue life of the initial tower is over 1000 years while for tower [B] with smaller wall thicknesses falls about 500 years. This shows that both towers are well designed, since the fatigue check is fulfilled. With a small reduction in shell thicknesses, the total mass of the tower is reduced about 20%, while the fatigue life is reduced by 50%.

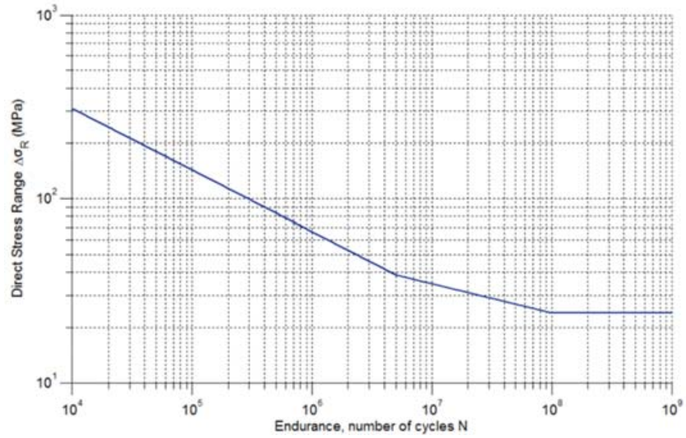

Figure 9. Direct Stress Range over number of cycles for the structural detail investigated

4.4 D Method - Damage accumulation calculation

The fatigue check of the structural details of the two towers using the D method is described in paragraph 2 of the present investigation. In this method simplified static analyses are conducted, since the loading of the structures is a single static load given by the manufacturer and the number of times (cycles) of occurrence of each load is given by the analytical equation provided by Eurocode [17]. The cumulative damage is calculated again using the Palmgren-Miner rule andits calculation is presented in Table 1. The annual cumulative damage for tower [A] is 0.0004 and for tower [B] is 0.0014. The fatigue check criterion is fulfilled for both towers and the fatigue life of the initial tower is 2500 years while for tower [B] with smaller wall thicknesses falls about 700 years.Again the fatigue check is fulfilled with this method but there is an overestimation of the fatigue life of the towers, and especially for tower [A]. The reduction in shell thicknesses, results in a total mass reduction of the tower of about 20%, while the fatigue life is reduced by 70%.

		Tower A			Tower B		
n_i	$\Delta S/S_k$	σ_i	N_i	n_i/N_i	σ_i	N_i	n_i/N_i
1	100.00	8.93	9.45E+08	1.06E-09	13.55	2.71E+08	3.69E-09
5	88.18	7.87	1.38E+09	3.63E-09	11.95	3.95E+08	1.27E-08
10	83.30	7.44	1.64E+09	6.11E-09	11.29	4.68E+08	2.14E-08
50	72.46	6.47	2.49E+09	2.01E-08	9.82	7.11E+08	7.03E-08
100	68.00	6.07	3.01E+09	3.33E-08	9.21	8.61E+08	1.16E-07
500	58.14	5.19	4.81E+09	1.04E-07	7.88	1.38E+09	3.63E-07
10^3	54.10	4.83	5.97E+09	1.67E-07	7.33	1.71E+09	5.85E-07
$5*10^3$	45.22	4.04	1.02E+10	4.89E-07	6.13	2.93E+09	1.71E-06
10^4	41.60	3.71	1.31E+10	7.61E-07	5.64	3.76E+09	2.66E-06
$5*10^4$	33.69	3.01	2.47E+10	2.02E-06	4.57	7.07E+09	7.07E-06
10^5	30.50	2.72	3.33E+10	3.00E-06	4.13	9.54E+09	1.05E-05
$5*10^5$	23.57	2.11	7.22E+10	6.93E-06	3.19	2.07E+09	2.42E-05
10^6	20.80	1.86	1.05E+11	9.52E-06	2.82	3.01E+10	3.33E-05
$5*10^6$	14.85	1.33	2.89E+11	1.73E-05	2.01	8.26E+10	6.05E-05
10^7	12.50	1.12	4.70E+15	2.13E-09	1.69	5.84E+10	1.71E-08
$5*10^7$	7.53	0.67	5.93E+16	8.44E-10	1.02	7.37E+10	6.79E-09
10^8	5.60	0.50	2.60E+17	3.84E-10	0.76	3.24E+10	3.09E-09
			D_d=	0.0004		D_d=	0.0014

Table1 : Damage accumulation calculation for method D

5 CONCLUSIVE REMARKS

The scope of the comparative study included in the present work isto assessthe fatigue life of welded connections between consequent steel shell parts of wind turbine towers. To this end, two towers of identical height and different shell thickness distribution are investigated and their fatigue life is assessed with two methods. The detailedanalysis method used in the present work introduces the use of time-varying loading instead of static, making the assessment of welded connections more accurate compared to the static loading that is widely used. The simplified method for fatigue assessment described in the European Standards is also presented in order to compare the results.In both analysis methods, tower [A] with greater shell thicknesses appears to have a longer fatigue life compared to tower [B]. The Eurocode simplified method named D in the present work overestimates a bit the fatigue life of the structures and especially for tower [A]. Tower [B] that is designed with 20% lower steel use still has a satisfactory fatigue life with both analysis methods, indicating that even with more detailed and demanding analyses, thinner shell thicknesses serve for economical structure construction along with satisfactory fatigue life. In common practice, structures subjected to cyclic loading with over 10^8 cycles, are constructed with increased tower shell thicknesses in order to limit the stress range variation under the cut-off limit. With the present study, it is shown that this increase in tower shell thicknesses is often leading to the construction of non-economical structures with unnecessarily increased fatigue life.

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