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WIND INDUCED FATIGUE IN TUBULAR STEEL WIND TURBINE TOWER WELDED JOINTS

Nafsika Stavridou[1], Evangelos Efthymiou[2] and Charalampos C. Baniotopoulos[1]

[1]School of Civil Engineering
University of Birmingham
Birmingham, B15 2TT, United Kingdom
e-mail: nstavrid@civil.auth.gr;c.baniotopoulos@bham.ac.uk, webpage: http://ccb.civil.auth.gr/

[2]Department of Civil Engineering
Aristotle University of Thessaloniki
Thessaloniki, GR-54124, Greece
e-mail: vefth@civil.auth.gr

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Abstract.*The constantly increasing need for sustainable energy production, leads to the construction of taller tubular steel wind turbine towers, which are the most common tower configuration. Recent catastrophic tower accidents, due to fatigue loading, challenges the engineering community towards more durable structures with minimized material consumption. The towers are designed to withstand a complex loading field, but cases of recent tower collapses happening in normal wind conditions and with shell stresses still in the elastic range, highlight the importance of fatigue phenomena and their consequences on the tower joints. The fabrication and mounting procedures applied for the construction of tubular steel wind turbine towers requests the realization of four types of welded joints. The four types of welded joints are: the circumferential welds between the subsequent tower parts, the longitudinal welds between the tower parts, the circumferential welds between the shell and the stiffening rings and the circumferential welds between the tower shell and the bottom flange. In order to assess the efficiency of such connections towards fatigue loading, two towers of identical height and variable shell thickness are compared. The calculation methodology applied is the damage accumulation method used for the fatigue life calculation of the structure, and important results on the importance of the tower joints on the overall tower behavior are derived.*

1 INTRODUCTION

Horizontal axis wind turbine towers are typically designed against ultimate, serviceability and fatigue loading. The governing loads taken into account at wind turbine tower design are according to Dimopoulos and Gantes [1], the bending moment and the lateral loading, both induced by the rotor operation. Tubular steel horizontal axis wind turbine tower structural detailing needs to be analyzed and designed with the use of refined real-size finite element models and advanced numerical techniques as proved in the work presented by Bazeos et al. [2] and Lavassas et al. [3]. Despite the fact that contemporary wind turbine structural analysis is performed by the combined use of macro- and micro-models, accidents continue to happen and sometimes total collapse of wind towers are observed. According to accident reports, 12% of the accidents is attributed to structural failure, while 6.8% is caused by material failure due to fatigue [4]. Wind tower post-collapse assessment offers valuable feedback on the accuracy of numerical modelling and its potential to predict the tower structural behavior. Lee and Bang achieved very accurate results on the numerical analysis of a collapsed wind tower using classic non-linear methods [5]. Repetto and Solari analyzing two slender wind towers designed to satisfy Structural Code criteria and taking into account damage accumulation due to wind induced loading, proved that even well designed structures can fail within the elastic stress range due to wind induced fatigue damage accumulation [6].

On-shore wind turbine towers are most frequently designed as steel tubular configurations with decreasing tower diameter from bottom to top and accordingly decreasing shell thickness from bottom to top. These tapered steel towers are manufactured in 20-25m long tapered sections that are mounted on-site by means of bolted flanges with the use of pre-tensioned bolts. These tower subsections are composed of rolled steel plates of approximately 2.5m wide, that are welded longitudinally in order to form rings, which are sequentially welded circumferentially in order to constitute the tower sub-sections. The tower is anchored to the concrete foundation with the use of pretensioned anchor bolts, connected the bottom tower flange with the foundation. The bottom tower sub-part is welded to the bottom flange that is positioned to the outer side of the tower with a circumferential butt weld. The mother material of wind turbine tower shells very rarely shows signs of fatigue failure, while as stated also in Nussbaumer et al. [7] design guidelines, discontinuities in the material and joint geometry, holes, bolts and welds lead to concentration of stresses and potential fatigue problems. Welds as a point of potential emergence of fatigue failure need to be carefully designed and thoroughly analyzed since according to Lacalle et al. [8], fatigue loading, inadequate weld design and poor welding execution is in most cases the reason of stress concentrations around the weld seams leading in most cases in noticeable fatigue damage at that point. The fatigue assessment calculation is performed in the above mentioned work with several analyses methodologies and the reliability of each method is questioned with the use of comparative results. In the work of Thanassoulas et al. [9] the rainflow method and Miner's rule have been used to assess the cumulative damage of the structure and to calculate the fatigue life of the structure concentrating on bolted connections of wind turbine towers. The significant contribution of this work is that in the procedure of fatigue life calculation of the structures, material nonlinearities, effects due to wind direction and design particularities due to different structural details are all taken into account. In the work presented by Jia [10], the application of a nonlinear dynamic analysis on the fatigue life assessment of tubular structures has been studied.

Having assessed from relevant bibliography that welded joints in wind turbine towers are potential areas of fatigue damage emergence and that a significant number of catastrophic accidents leading to wind turbine tower collapse are provoked by fatigue phenomena, led the present scientific work to focus on enlightening the detailed analysis of welded joints. The tool used to assess the performance of welded joints is the comparison of the fatigue life of two identical tubular steel towers whose shell thickness distribution differs along their height, based on the calculation of damage accumulation of four different types of welded joints. The first is the circumferential weld connecting the tower shell to the bottom tower flange, named as detail B hereafter, the second is the circumferential weld connecting consequent tower rings in order to constitute a tower module, named as detail C hereafter, the third is the longitudinal weld connecting the curved plate edges in order to constitute a tower ring, named as detail L hereafterand the fourth is the circumferential weld connecting the tower shell to the circular flange, named as detail F. For the implementation of the above mention method, finite element analysis of the towers is performed with the aid of the commercial software ABAQUS [11]. The fatigue life of the structures is estimated with the implementation of the damage accumulation method and more specifically the Palmgren-Miner rule. The comparative results of the tower models and the different welded joints types are presented and special remarks on the effect of the fatigue design on tower shell thickness are made.

2 WIND TURBINE TOWER FATIGUE ANALYSIS

2.1 Definition and phenomenon

The phenomenon stresses be repeatedly applied and relaxed by the resonant motion of the rotor due to the wind flow in wind turbine towers, causing cracks due to excessive stress concentration, can be considered as damage accumulation due to fatigue. When the steel wind turbine tower shell is repeatedly bent back and forth, material damage at the areas of great stress concentration is observed with crack initiation. The phenomenon of fatigue that is initially expressed with the emergence of small cracks at a certain structure region of great stress concentration continues with the propagation of the cracks leading to structural failure and possible consequence a total structure collapse. From post-collapse structure analyses, it is observed that collapses happened without any prior notice and with stress amplitude being still in the elastic range of the material. This can be explained, since cyclic loading provokes steel hardening that makes the material brittle, leads to the emergence of cracks that expand with the cyclic loading and the material fails without prior occurrence of large deformations. The mother material very rarely presents fatigue failure, while spots with discontinuities, holes, thickness changes, welds, bolts, etc. are more vulnerable to damaged due to alternate loading.

2.2 Methodology

According to EN 1993-1-9 [12], one of the methods incorporated for the fatigue assessment of structures is the damage accumulation method. With this methodology, the structural details that need to be checked against fatigue are selected and the stress difference observed in certain loading cycles is calculated. The loading events that can be used for this method, as analyzed in Annex A of EN 1993-1-9 [12] are based on prior knowledge obtained from similar structures, in order to represent a credible upper bound of the expected loading that the structure is going to be subjected to. Since the principal loading that wind towers are subjected to is the wind, wind loading histories from similar size prototype structures or site data obtained from implemented structures can be used for the fatigue life assessment and design of similar size future towers. Due to the fact that real wind loading data take a lot of time to be collected, are quite hard to manipulate and may also differ even between seemingly same structures and geographical areas, in the present scientific work the use of artificial loading histories is proposed. These artificial time histories are produced by the NREL [13] and NWTC [14] open access software: TurbSim [15], Aerodyn [16] and FAST [17]. The production of artificial loading is faster and accurate since software from aerodynamics is used. The wind loading histories produced by the above mentioned software are calculated at the tower hub height, for a given terrain roughness category and according to the Kaimal frequency spectrum. From the 6 loading sequences produced by the software only the two principal loadings are taken into account in the present study; the main bending moment and the lateral loading caused by the rotor function at the upwind direction.

In the present scientific study, the damage accumulation method with determination of fatigue load parameters and verification formats described in EN 1993-1-9 - Annex A [12] is implemented in four types of structural details existing in typical tubular wind turbine towers. Following this methodology, a preliminary static analysis is primarily conducted in order to determine the areas of the structure that are more vulnerable to show fatigue induced failure. The basis of this methodology is to determine the stress history developed in the structural details under consideration where great stress concentration is observed. This stress history, as mentioned in the Eurocode specifications, is determined from measurements on similar structures or from dynamic calculations of the structural response. In the case of the present work, the stress histories are determined from the artificial wind loading events applied on the numerical model. There are multiple loading histories applied in each tower, covering the range of operational mean wind speeds. After obtaining the stress histories developed at the structural details under consideration, the rainflow cycle counting method is chosen to convert the complex stress time histories into stress range spectra. The stress range spectra for each mean wind speed are combined in order to constitute the spectrum of amplitudes of stress cycles in one year. Having associated the detail category under investigation with the relevant S-N curve provided in Eurocode tables [12], the damage of the structures can be calculated by applying the linear damage accumulation method, called Palmgren-Miner rule. The cumulated damage is described by equation (1):

$$D_d = \sum_{i}^{n} \frac{n_{Ei}}{N_i} \tag{1}$$

where:

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

N_i= The endurance (in cycles) obtained from the factored curve.

3 NUMERICAL MODELS

3.1 Model description

The wind turbine tower models used for the comparative analysis of the fatigue life of certain structural details, are simulations of real implemented structures. The towers share the same height of 76.15 meters and comply with certain restrictions concerning the maximum tower diameter and the maximum tower part length due to transportation difficulties. 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 thickness distribution along the height of each tower can be observed in Fig. 1. 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 [18].

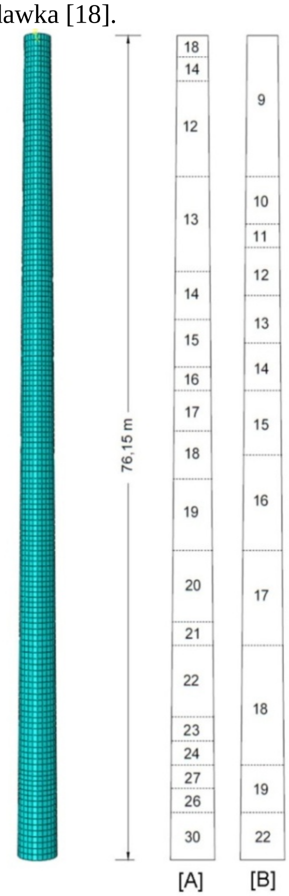

Figure 1. Shell thicknesses of investigated towers [A] and [B]

3.2 Loads incorporated

For the preliminary static analysis, the fatigue loads taken into account are provided by the manufacturers and are the fatigue loads from the motion of rotor. These data are the horizontal force of F=75,5kN and the horizontal axis moment of M=1091kNm. The tower weight is ignored for the fatigue analysis.

For the analyses with the artificial loading histories, the loads are calculated from NREL software and as already mentioned, only the two principal loading histories on the top of the tower are taken into account: the horizontal force history and the bending moment history. The concentrated loads are applied at the top of the tower to a reference point taking into account the eccentricity of the rotor position. The point where the loads are applied is positioned at the top of the tower having the center of gravity shifted horizontally +0:725m from the axis of the tower and vertically +0:50m above the upper flange level (+76:15m). The time-histories applied at the top of the tower are for variable hub height wind speeds in order to cover all the spectrum of operational winds.

4 RESULTS

4.1 Stress range histograms

The artificial time histories for shear and moment loading are implemented at the hub height of the two towers under investigation. These data are obtained from the free software Turbsim, Fast and Aerodyn for all the wind speeds taken into consideration. The four structural details that are going to be examined are preselected from the results of the initial static analysis. After the numerical analysis of the two towers, the normal stresses σ_{zz} of the tower shell at all four structural details under consideration are obtained, since the tensile stresses are the ruling factor leading to fatigue failure. In Fig.2 and Fig. 3 the stress histories σ_{zz} at the selected detail type C of the towers [A] and [B] respectively are presented for the various wind speeds.

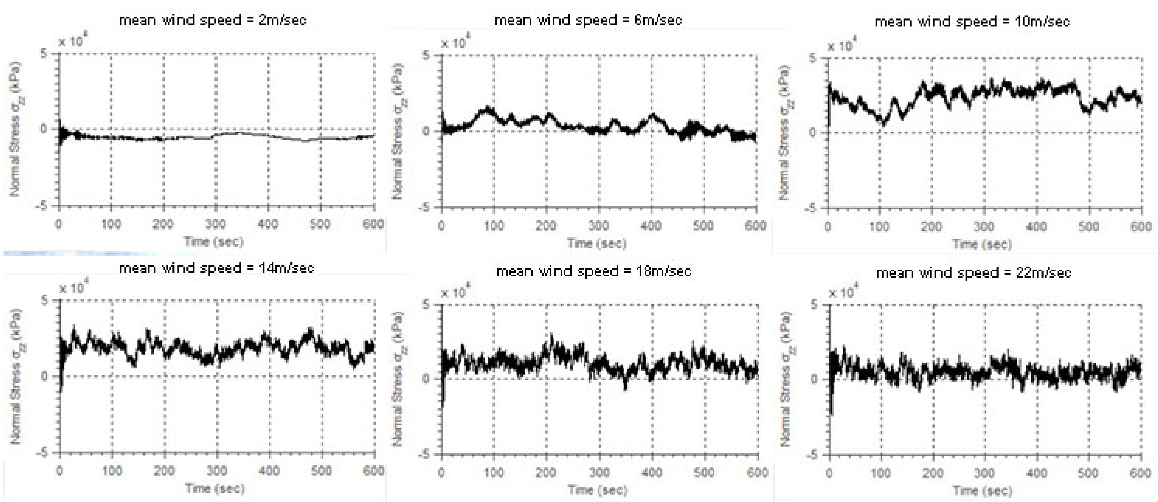

Figure 2. Detail Type C – Tower [A] stress histories

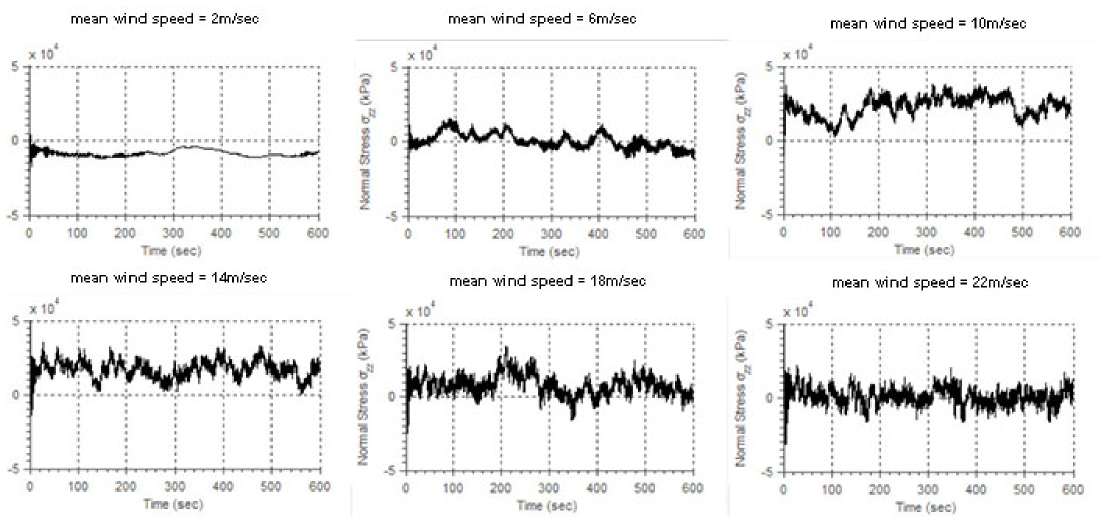

Figure 3. Detail Type C – Tower [B] stress histories

4.2 Cycle counting

The stress diagrams obtained by the numerical analysis are time related and great variability of frequencies and ranges are observed. These complex stress time histories need to be converted to histograms of number of cycles and stress levels in order for the damage accumulation method to be applied. This is realized in practice by converting the complex 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 study the rainflow cycle counting method is selected and it is applied through MATLAB software. For the four spots of the towers that are already predefined, the stress level histograms are produced for each one of the six wind speed time histories as shown in Fig.4 and Fig.5. 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.

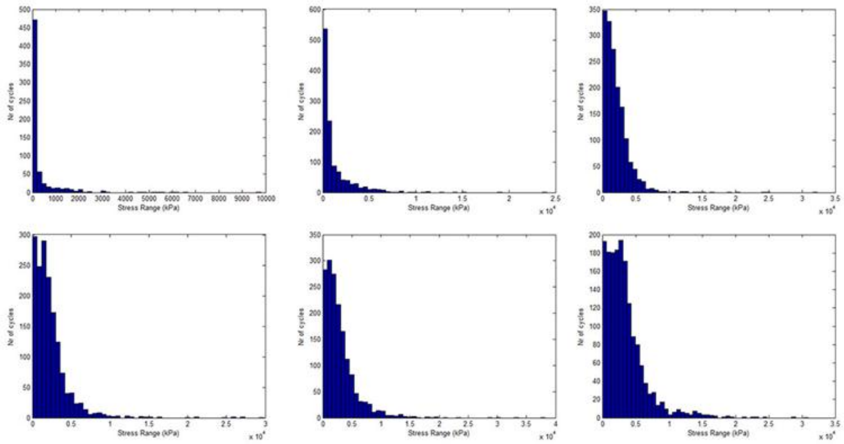

Figure 2. Detail Type C – Tower [A] stress histories

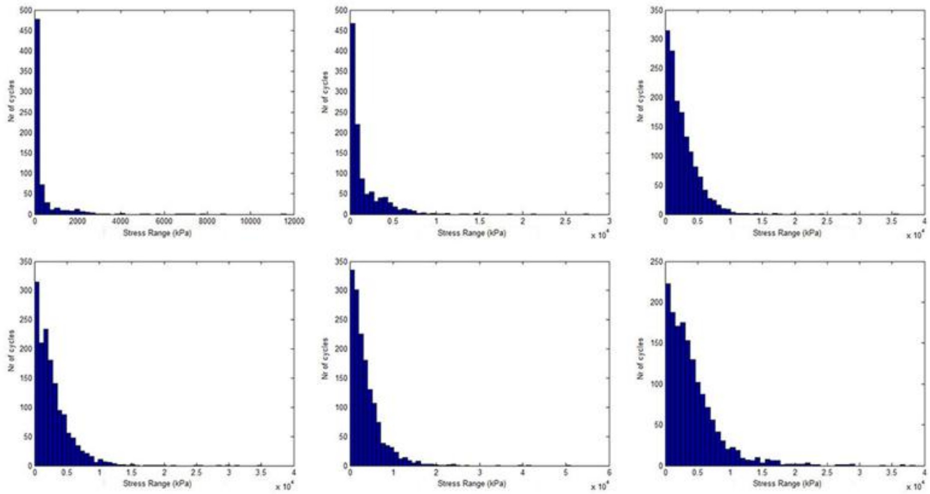

Figure 3. Detail Type C – Tower [B] stress histories

4.3 Damage accumulation calculation

The above presented stress range histograms show 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 [19] the distribution of wind speeds over an extended period of time is given by the Rayleigh or the Weibull distribution. In the present study, the Rayleigh distribution is used and according to the Rayleigh distribution the cumulative probability function is given by eqn (2).

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

where Vave = 0.2*Vref and for wind turbine class II [20]Vref = 42.5 m/sec.

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 structural detail and for both towers, is presented in Fig.4-7.

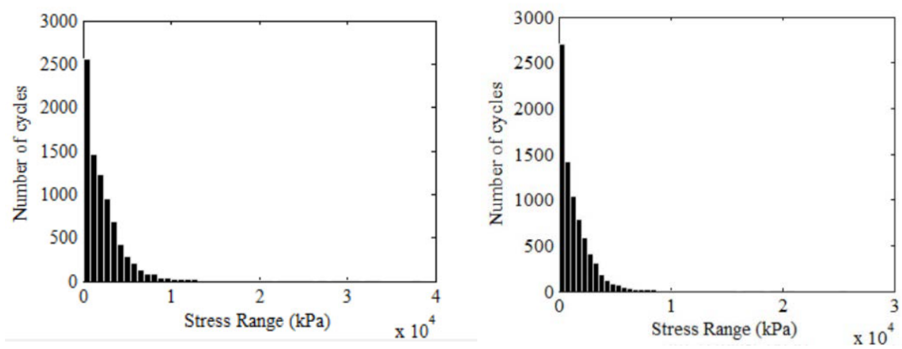

Figure 4. Annual Stress Range Spectra - Detail Type C – Tower [A] and Tower [B]

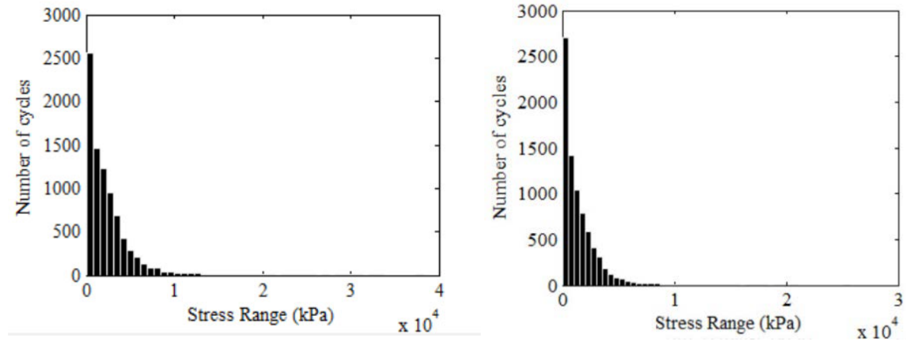

Figure 5. Annual Stress Range Spectra - Detail Type L – Tower [A] and Tower [B]

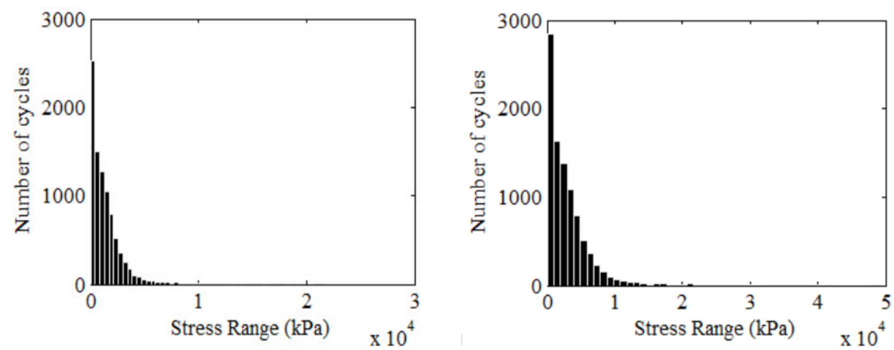

Figure 6. Annual Stress Range Spectra - Detail Type F – Tower [A] and Tower [B]

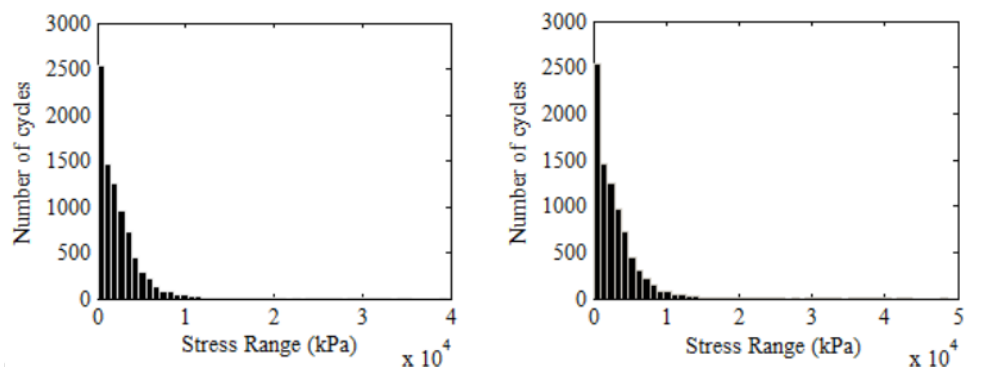

Figure 7. Annual Stress Range Spectra - Detail Type B – Tower [A] and Tower [B]

Damage due to fatigue is cumulative and the linear damage accumulation assumption is made where stress ranges, occurring n_i times, result in a partial damage that is represented by a ratio n_i/N_i where N_i is the number of cycles to failure. The accumulating damage D_d and its reciprocal value for each structural detail of each tower is presented in Table 1. The most common detail categories, according to EN 1993-1-9 [12] have been classified according to their fatigue stress capacity and the factor capacity curve for each one of those can be formulated. The circumferential weld between consequent tower parts has a fatigue stress capacity of $\Delta\sigma_c$=80 MPa and the corresponding factored capacity curve is presented in Fig. 8.

For the fatigue check of the structural details of the two towers examining all four types of structural details, the cumulative damage has to comply with the criterion of Dd<1.0. The annual cumulative damage for each detail and each tower is presented in Table 1. It is shown that both towers are well designed, since the fatigue check is fulfilled. It is significant that with a small reduction in shell thicknesses, the total mass of the tower is reduced about 20%, while the fatigue life is reduced by about 50%.

	Tower [A]		Tower [B]	
Detail Type	D_d	Fatigue Life	D_d	Fatigue life
C (weld)	0.0007	1428 years	0.0019	526 years
L (long. Weld)	0.0008	1250 years	0.0021	476 years
F (flange)	0.0028	357 years	0.0082	122 years
B (bottom weld)	0.0124	80 years	0.0323	35 years

Table1 : Fatigue life of structural details of towers [A] and [B]

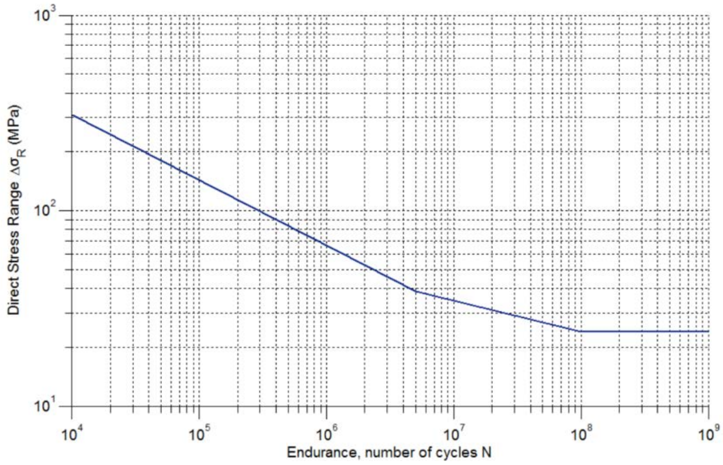

Figure 8. Direct Stress Range over number of cycles for detail type C

5 CONCLUSIONS

The scope of the comparative study included in the present work is to assess the 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 by examining four types of welded joints. The detailed analysis 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. Tower [A] with greater shell thicknesses appears to have a longer fatigue life compared to tower [B] in all details' analyses, showing the significance of the shell thickness in the fatigue life calculation of steel shells. Tower [B] that is designed with 20% lower steel use still has a satisfactory fatigue life, indicating that with the performance of more detailed and demanding analyses, thinner shell thickness use can be achieved and more economical structures can be designed. 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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