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Structural analysis and optimal design of steel lattice wind turbine towers

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Abstract (150 words)

Wind energy has shown its dominance among sustainable energy production means by the accelerating rise in total installed capacity and size increase of wind energy structures. Taking into consideration the fact that the supporting structure of onshore wind power generators constitutes approximately one third of the initial construction cost, structural optimization of the tower is considered crucial towards capital expenditures minimization during construction. Contemporary energy needs employ the construction of constantly taller and more powerful wind power converters, whose robust design in parallel to initial capital expenditures compression is not to be neglected.

The dominant structural configuration for onshore wind power generators is the tapered steel tower, but lattice ones with the used of enhanced special cross-sections can be a rather promising solution towards material use economy. The present paper addresses the structural performance and optimization of tubular and lattice steel wind turbine towers, examining alternative configuration solutions for a given height and rotor characteristics. The finite element software Abaqus has been used for the implementation of the structural models and an algorithm has been elaborated in Mathematica software in order to allow for the cross-section use optimization in the case of lattice towers.

Keywords chosen from ICE Publishing list

Steel Structures; Mathematical modelling; Buildings, structures & design

List of notation

F	is the loading condition applied on the tower
W_N	the rotor self-weight
V_R	shear force due to rotor's function
M_R	moment due to rotor's function
σ_{eng}	engineering stress
ε_{eng}	engineering strain
E	is the modulus of elasticity
σ_{true}	true plastic stress
ε_{ln}^{pl}	is the true plastic strain
$N_{b,Rd}$	is the design buckling resistance of a member
N_{Ed}	is the design axial force
D	is the member diameter
t	is the member's thickness
ε	is a coefficient depending on f_y

f_y	is yield strength for steel
$N_{45}(z)$	is the axial force when shear acts in the direction of the diagonal of the tower
$M(z)$	is the bending moment of a vertical cantilever
$B(z)$	is a geometrical parameter correlating the top and base width
ϕ_d	is the angle of the leg to the horizontal plane
B_{base}^{ch}	is the characteristic base width that keeps the axial force of the legs constant
B_{top}	is the tower top width
M_{top}	is the moment at the top of the tower
H	is the tower height
F_{top}^H	is the horizontal force acting at the top of the tower
B_{base}	is the tower base width
μ	is the parameter that determines the deviation from the characteristic base width
FBT	face bracing truss
P	integration point where stress is calculated

1 Introduction

2 The use of sustainable energy production means is considered crucial nowadays in order to
3 preserve the current climate conditions and limit the use of fossil fuels that are finite and their
4 emissions are exacerbating the greenhouse effect. To this end, the renewable energy policy
5 framework established for United Kingdom, sets a target of sourcing 15% of final energy
6 consumption from renewables by 2020 (IEA, 2017). Due to the infinite nature and great potential
7 of wind energy, its applications have shown remarkable evolution and expansion the last ten
8 years. Indicatively the power capacity of wind parks installed in Europe has tripled from about
9 50 GW in 2005 to over 150 GW in 2016 (Pineda and Tandiou, 2016). Aiming to more robust
10 onshore wind structures with minimized design and material costs, special attention has to be
11 paid in the design of the tower, which constitutes more than 25% of the initial wind turbine cost
12 (Blanco, 2009). In the present work, the structural configuration of onshore wind turbine towers
13 is investigated and analysed in terms of structural behaviour and material use. The aim of the
14 investigation is to minimize the wind tower initial construction cost by means of limiting the
15 material use both in the upper structure and the foundation.

16
17 The three commonest tower configurations for horizontal axis onshore wind converters are: the
18 tubular steel tower, the lattice tower and the hybrid tower; meaning a combination of a truss
19 structure for the lower part and a tube of the upper one. The prevailing structural configuration
20 when examining the total installed wind converters is the steel cylindrical shell tower due to its
21 robust structural detailing, easier mounting and limited labour required on site. Cylindrical shells
22 towers due to their geometry offer the advantage of carrying great loads with small shell
23 thicknesses. The tubular tower is composed of subsequent conical parts manufactured in the
24 factory, transported on site and mounted to their final positions by means of bolted flanges with
25 the use of pretensioned bolts (Lavassas *et al.*, 2003). The governing loads acting on the tower
26 are the wind pressure up the tower height, the moment and the lateral load due to the rotor
27 operation and the vertical load that is equal to the rotor weight. The structural analysis of wind
28 turbine towers is considered of high importance since failure in such projects has great
29 economical, structural and safety losses. The investigation of the buckling behaviour of
30 cylindrical shells has been founded by the research work, both numerical and experimental,

conducted in the past by a plethora of research groups like Timoshenko and Gere (1961), Bazant and Cedolin (2010) and Teng and Rotter (2004). Tubular steel towers belong to the general research area of cylindrical shells under combined loading and specialized research on their structural analysis and response towards wind loading has been conducted in the work of Baniotopoulos *et al.* (2010). Lee and Bang (2013) performed an elaborated numerical analysis in order to simulate the collapse of a previously installed wind turbine tower by a lightning strike whilst valuable work has been conducted in the analysis of wind turbine towers against seismic loading (Arasu *et al.*, 2011; Nuta *et al.*, 2011). Contemporary tower design of constantly greater height, requires increase in the thickness of steel shells; thus increased material used for their construction. Several groups have worked on the introduction of stiffeners inside tubular structures in order to ameliorate their structural performance while maintaining the overall shell thickness at the same level (Dimopoulos and Gantes, 2012; Rebelo *et al.*, 2012a; Rebelo *et al.*, 2012b, Stavridou *et al.*, 2013; Stavridou *et al.*, 2015).

Towards taking advantage of the higher energy potential at greater heights, contemporary wind energy structures are taller and with greater energy production capacity. Since, the tubular tower configuration functions as a simple cantilever structure, the taller the structure the bigger the bottom diameter needed in order to sustain the great resisting moment at the base. Certain transportation limitations (e.g. Bridge span at highways etc.) prevent the towers from having greater diameters and longer subparts. In cases where transportation limitations apply, the lattice tower appears as an appealing solution for wind turbine towers. Their structural behaviour has been investigated thoroughly by various research teams; against wind loading (Holmes, 1994; Holmes, 1996a; Holmes, 1996b; Harikrishna *et al.*, 1999; Carril, 2003) and against seismic loading (Khedr, 1998; Khedr *et al.*, 2000) while their dynamic response has been investigated in the work of Madugula (2001). Lattice towers have been extensively implemented on telecommunication masts and their structural behaviour in this application has been investigated explicitly (Tsitlakidou *et al.*, 2005; Efthymiou *et al.*, 2009). The implementation of lattice towers on offshore and onshore wind turbines has just been investigated the past years (Zwick *et al.*, 2012; Long *et al.*, 2012; Long and Geir, 2012; Gencturk *et al.*, 2012) and still their optimal design has not yet been studied. This offers a first class opportunity for high-quality

original research conducted in the present work. Conventional lattice towers are constructed mostly with the use of standard cross sections fabricated in the factory and mounted on site. The scale of the lattice towers which are able to support the rotor of a wind converter leads to cross-sections that are well outside the range of standard industrial profiles. A lattice tower that is capable of accommodating the nacelle has the form of a truncated cone with a polygon or square base. The tower is a statically determinate lattice structure composed of a number of discrete structural sub-systems; the legs, the bracing trusses on the faces, horizontal braces and secondary bracings arranged inside the plane of the face bracing trusses. The aforementioned structural subsystems have a particular role in the load transfer mechanism that develops inside a lattice tower and since the tower is a statically determinate structure, the axial stresses of the legs and the bracings can be determined by closed form expressions. The present paper addresses the stability performance comparison of a tubular steel wind turbine tower and a lattice wind turbine tower of the same height and with the same loading applied at the hub height. It examines the performance of each tower while attempting to minimize the total material used maintaining its endurance and robustness.

2. Structural Analysis and Numerical modelling

The numerical investigation for the two tower configurations has been performed with the general purpose finite element software ABAQUS (Hibbitt *et al.*, 2001) in order to assess the structural response of the tubular and lattice tower configuration under combined loading. The tubular tower model is validated in comparison with previously performed work of the research group, while the lattice tower model has been validated in comparison with a parametric study performed in Mathematica Software (Wolfram, 1999) with the implementation of the design code and analytical equations.

2.1 Tubular Tower

The tower used for the current investigation has been examined in the HISTWIN project (Veljkovic *et al.*, 2006) and is a tubular steel wind turbine tower implemented for onshore structures. It consists of three subparts to facilitate its transportation on site, with lengths of 21.8m, 26.6m and 27.8m respectively counting from bottom to top and completing the total

tower height of 76.15m. The configuration is a truncated cone with varying diameter and thickness along its height as it can be observed in Figure 1. The tower shell is fixed to the bottom flange which is embedded into the reinforced concrete foundation by means of prestressed anchor bolts; therefore the tower model functions as a cantilever structure and is considered fixed at its bottom.

For the modelled geometries the four-noded doubly curved shell element S4R with reduced integration and finite membrane strains was adopted as it has shown to perform well in similar structures (Stavridou *et al.*, 2015; Lee and Bang, 2013). For the selected mesh, a convergence study has been performed prior to the analysis. The design loads that the wind tower has to sustain are: the self-weight of the rotor (W_N), the shear force due to the rotor's function (V_R), the moment due to the rotor's function and the wind loading (M_R) as described in Equation 1.

$$F = W_N + V_R + M_R$$

1.

In the present study, focus is on the overall response of the tower against normal operating conditions, therefore fatigue, seismic and icing loading are not taken into account. The weight of the platforms and the ancillary equipment is neglected. The concentrated loads that apply on the tower at the connection of the tower to the nacelle, are modelled as concentrated loads applied at a reference point (RP), taking into account the horizontal ($e_x = +0.725\text{m}$) and vertical ($e_z = +0.5\text{m}$) eccentricity of the rotor position as schematically shown in Figure 2. The steel grade used in the present work is S355 and the material properties incorporated into the finite element models were based on the coupon tests performed in HISTWIN project (Veljkovic *et al.*, 2006), in the form of an elastic-plastic multi linear curve with the von Mises yield criterion and isotropic hardening. ABAQUS requires the material properties to be input in the form of a multilinear true stress-true plastic strain curve, therefore the measured engineering stress-strain curves are converted into the true stress-true plastic strain curves by means of Equations 2 and 3, where σ_{eng} is the engineering stress, ε_{eng} is the engineering strain, E is the Young's modulus, σ_{true} is the true stress and ε_{ln}^{pl} is the logarithmic plastic strain.

$$\sigma_{true} = \sigma_{eng} (1 + \varepsilon_{eng})$$

2.

$$\varepsilon_{ln}^{pl} = \ln(1 + \varepsilon_{eng}) - \frac{\sigma_{true}}{E}$$

3.

Hence, the material law introduced in the model consists of 3 branches as observed in Figure 3: the first is an almost constant plateau until 0.011 true strain, the second is a steep branch until 0.0528 true strain and 539 MPa true stress and the third is a smooth branch until ultimate plastic strain 0.174% and ultimate true stress 612 MPa..

For computational efficiency, the support of the structure and the top plate of the tower are introduced to the model with the use of appropriate boundary conditions and constraints. At the foundation, the degrees of freedom of all the nodes are constrained to the degrees of freedom of a control point node through kinematic coupling constraints replicating the theoretical lack of deformation of the bottom diameter due to the attachment to the tower foundation. The top plate where the loads of the nacelle are acting on the tower, is simulated with a reference point at the centroid of the circle (master point) bonded with the nodes at the perimeter of the tower (secondary points) by means of kinematic coupling constraints. This simulation method constrains the nodes of the tower perimeter to the rigid body motion of a single node, the one at the centroid. The loads are acting on the master point and all the nodes of the top perimeter are acting as a diaphragm, sharing the same displacement with the master point.

Local geometric imperfections are introduced into the models in the form of the elastic buckling mode shape that corresponds to the lowest eigenvalue. As Koiter (1945) first introduced and Speicher and Saal (1991) implemented on shell structures, the equivalent imperfection most suitable for the investigation of steel shells is of the same form as the first bifurcation mode and the imperfection amplitude is calculated according to paragraph 8.7 of Eurocode EN1993-1-6 (2007). A non-linear static analysis taking into account both material and geometric nonlinearities was implemented in the present work in order to trace the full load-deformation response path of the modelled tower.

2.2 Lattice Tower

The lattice tower investigated shares the same height as the tubular one and bears the same loads and nacelle size. The lattice tower that accommodates the nacelle at its top has in the present investigation, the form of a truncated cone with a square base. The tower under discussion like most truss systems, is a statically determinate structure composed of discrete structural sub-systems; the legs, the bracing trusses on the faces and the horizontal braces as described in Figure 4. Each of the aforesaid structural sub-systems have distinct roles in the load transfer mechanism of the tower. The legs transfer the axial load from the nacelle weight and the moment deriving from the horizontal shear and top-moment imported to the lattice structure at its top by the connecting plate of the nacelle. The face bracing truss undertakes the transfer of the horizontal shear and the horizontal braces take the out-of-plane buckling of the face bracing truss (FBT) elements. Being a structurally determinate system means that the axial stresses of the tower legs and the FBT elements can be determined by closed form expressions.

The problem of the design of a cross section in a certain member calls for the choice of a cylindrical cross-section that will result to a $N_{b,Rd}$ equal to the design axial force N_{Ed} according to Eurocode EN1993-1-1 (2005). The problem of determining the diameter (D) and the thickness (t) of the most economical cross section can be solved by keeping the D/t ratio to the limiting value of Class-III cross sections i.e. $90\varepsilon^2$ where ε is given by Equation 4.

$$\varepsilon = (235 + f_y)^{0.5}$$

4.

The whole issue of structural optimization of the given problem is treated by developing a specialized script in Mathematica software, which finds the diameter of the tubular cross-section that fulfils the buckling resistance and minimizes the tower weight.

On the optimization of the FBT and the legs some geometrical parameters have to be taken into consideration. Since the form of the tower is conical, among the most popular bracing geometries, the V shaped FBT appears more favourable as it keeps the total length of the diagonals less than the X brace and the inverted V brace. A special subroutine is dedicated in

the present investigation, to investigate the V brace angle that leads to the minimization of the total FBT weight. The angle of the V braces determines in addition the segments that the tower is split to and therefore the buckling length of the tower legs. The normal force that develops in the legs of a tower with a square base is derived from equilibrium considerations and the governing situation for the legs appears when the top shear acts in the direction of the diagonal of the tower. Hence, the axial force is given by Equation 5:

$$N_{45}(z) = \frac{M(z)}{b(z)\sqrt{2}} \times \frac{1}{\cos(\pi/2 - \phi_d)}$$

5.

Where $M(z)$ is the bending moment of a vertical cantilever, $b(z)$ is a geometrical parameter correlating the top and base width and ϕ_d is the angle of the leg to the horizontal plane. Using as a reference case the one with constant leg cross-section from bottom to top and therefore constant axial force, there is a characteristic base width given by Equation 6 that keeps the axial force constant.

$$B_{base}^{ch} = B_{top} \frac{M_{top} + H \times F_{top}^H}{M_{top}}$$

6.

Where B_{top} is the top width, M_{top} is the moment at the top of the tower, H is the tower height and F_{top}^H is the horizontal force acting at the top of the tower

Taking all the above into account, the optimization problem of the lattice tower is confined to the search space defined by the two independent variables: B_{top} and μ . The variable μ is a non-dimensional parameter that determines the deviation of the base width from the characteristic and is given by Equation 7.

$$\mu = \frac{B_{base}}{B_{base}^{ch}}$$

7.

The total weight of the tower is determined by the buckling checks described in Eurocode EN1993-1-1 (2005), which is a highly non-linear procedure and is greatly influenced by the

buckling length, which is controlled in the present tower type by the introduction of secondary bracing. Therefore the optimal weight does not come as a result of deriving equation 6, but needs an iterative procedure to take into account and counterbalance the two antagonistic factors that determine the optimal tower design; (a) the parallel increase of the leg axial force along with the reduction of the face bracing weight, when closing the distance between the legs and (b) the parallel reduction of the leg axial force along with the total length and slenderness increase of the V braces, when increasing the distance between the legs

The script performs a two-dimensional search in order to assess the variation of the total tower weight with respect to the B_{top} and μ using a successive iterations.

The tower model with minimum weight is verified with the use of ABAQUS (Hibbitt *et al.*, 2001) software. The tower is of 76.15m height and the number of segments of the FBTs is determined from the iterative procedure that optimizes the tower weight with the V-brace angle diversity. All the model elements are modelled with T3D2 truss elements and the tower joints are introduced to the model with the use of with the use of appropriate boundary conditions and constraints. At the foundation, the degrees of freedom of all the nodes are constrained, since the tower is considered fixed to its foundation. The design procedure introduced in the script is verified with the ABAQUS model achieving very satisfying results.

3. Results

3.1 Tubular Tower Results Analysis

3.1.1 Eigenvalue analysis

An initial bifurcation analysis is imperative for the calculation of the critical elastic buckling mode shape and the critical buckling load of the structure. The buckling load is critical in order to understand the maximum load that the tower can sustain before failure, while obtaining the eigenmode shapes of the structure is important for the introduction of initial imperfections in the numerical analysis with the use of the respective amplitude factor. The first three eigenmodes of the tower are obtained as shown in Figure 5 and 6. The short wavelength buckling shape that prevails in all the eigenmodes is characteristic of dominant bending moment and compressive

forces acting on the structure. The critical buckling load for each eigenmode is presented in Table 1.

3.1.2 MNA and GMNIA analyses

Regarding the tubular tower, the material nonlinear analysis results are presented in Figure 7, showing the tower response towards combined loading. The material properties are considered in terms of plastic true stress and plastic true strain as described above and the stress concentration is observed at the top 1/3 of the tower as expected. When taking into account for the initial imperfections, the 1st eigenmode deformed shape is used with the amplification factor proposed in Eurocode EN 1993-1-6 (2007). The short wavelength buckles of the first eigenmode are initializing more complicated deformations like “ovalization” of the cross-section and long wavelength buckles of the circumference of the shell, as observed in the results of the geometric and material nonlinear imperfection analysis in Figure 8. For both analysis methods, concentration of high von Mises stresses appears at the upper 1/3 of the structure. The tower shell thickness distribution is optimized and in both analysis methods the von Mises stresses at the critical point are at the limit of the yielding, as it can be observed from Figure 9. Points 22-24 and 1-5 are the leeward side of the tower where high stresses are observed, forming the potential buckling zone. The sides of the tower (points 6-9 and 18-21) have very low stresses and the windward side (points 10-17) of the tower has stresses that towards the final load step have higher values but are below yielding. The ovalization of the tube when taking into account the initial imperfections (Figure 8) indicates their immense importance to the final failure of the structure. The failure load of the structure appears to be one fourth of the load indicated by the eigenvalue analysis. This phenomenon of shells that buckle at a lower ultimate load than the one that is shown by the eigenvalue analysis is expected and has been described extensively in the cylindrical shells theory. As far as the total tower mass is concerned, the optimized shell tower weighs 127.215 tn, with the three tower subparts weighing 54.54 tn, 44.817 tn and 27.858 tn respectively from bottom to top.

3.2 Lattice Tower Results Analysis

The combinations of B_{top} and μ used to build the design cases are B_{top} [350mm – 1000mm] and μ [0.65 – 1.3] and the total number of cases investigated for the tower optimization are 126. All the lattice tower cases share the same height with the tubular one and undertake the same loading as the tubular one.

The weight of the tower in correlation to the non-dimensional parameter μ for a given B_{top} is presented in Figures 10 and 11. The higher the top width, the greater the tower weight. For narrow top widths, from 350 to 500 mm, all the lattice towers are lighter than the tubular one independently of μ . When the tower top is wider, the weight of the lattice tower increases almost exponentially with the increase of μ . In Figures 12-24 the weight of the tower is displayed with regards to the top width for all the given levels of divergence of the base width from the optimal (i.e. $\mu = 0.6$ corresponds to the weight of the tower for $\mu = 0.6$).

The total weight for the optimal lattice tower solution is 77.47 tn and is given for of B_{top} equal to 800mm and μ equal to 0.6.

As described above, the braces implemented are of V type, the V brace angle is set to 45 degrees and the brace angle is the driving criterion for the split of the overall tower to its subparts. The optimal tower is split in 5 subparts with heights described in Table 2. As the tower is symmetric and the wind load can come from any direction, in each tower subpart the same type of elements are selected to have the same cross-sections. In our case circular hollow cross-sections are used and the optimal tower design is presented in Table 3.

The tower is almost 30 % of the initial construction cost of a wind turbine. Therefore, the reduction of the material used is of great importance in the economical aspect. As it can be observed from the comparison of the results of the two tower solutions, the total tower weight, using the lattice solution, is reduced by almost 40 %. Taking also into account the fact that in terms of transportation and in-situ construction, the lattice solution is advantageous in regards to flexibility in transportation and easiness in mounting, the lattice solution should be taken into consideration for the construction of contemporary wind turbines.

4. Conclusions

The present study investigates the potential of substituting tubular wind turbine towers with lattice ones with the project of minimizing the total structure weight while maintaining the structure load bearing capacity. With the prospect of constructing even taller structures, the minimization of the total material use is of great importance along with the transportation advantages that truss structures exhibit over the tubular ones. Cylindrical shells have been proven to be robust enough, offering higher capacity to the structure, but substantially greater amounts of material use. On the other hand, lattice structures when using the appropriate cross-sections for construction of tall structures are proved to be able to sustain great loads with minimum initial material weight. An additional advantage that lattice structures offer is the lower bending moment transferred to the foundation, which facilitates the concrete foundation design and minimizes its construction cost. In the present comparative study, a wind turbine of 76.15 meters height has been designed using both the tubular and the lattice solution. An iterative procedure has been used to trace the optimal lattice tower configuration, which minimizes the total tower weight. The design loads are identical in both types and the respective solutions are proven robust enough. In terms of total weight, the lattice tower is 40% lighter, minimizing by almost 15% the total initial construction cost. The advantages that the lattice solution offers in terms of transportation and fabrication, along with the flexibility of its configuration may lead to great and advantageous changes in the configuration concept in wind turbine tower design.

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References

Arasu PV, Sagayaraj D and Gowrishankar J (2011) Seismic analysis of a wind turbine steel tower. In *Proceedings of Simulation Driven Innovation, L&T Integrated Engineering Services, HTC, Altair-2011 HyperWorks Technology Conference*. Pune, India, pp. 4-5.

319 Baniotopoulos CC, Borri C and Stathopoulos T (2010) *Environmental Wind Engineering and*
320 *Design of Wind Energy Structures*. Springer, Wien, Austria.

321 Bazant Z and Cedolin L (2010) *Stability of Structures: Elastic, Inelastic, Fracture and Damage*
322 *Theories*. World Scientific Publishing, Singapore.

323 Blanco MI (2009) The economics of wind energy. *Renewable and Sustainable Energy Reviews*
324 13(6-7): 1372–1382.

325 Carril JrCF, Isyumov N and Brasil RM (2003) Experimental study of the wind forces on
326 rectangular latticed communication towers with antennas. *Journal of Wind Engineering and*
327 *Industrial Aerodynamics* 91(8): 1007-1022.

328 Dimopoulos C and Gantes C (2012) Experimental investigation of buckling of wind turbine tower
329 cylindrical shells with opening and stiffening under bending. *Thin Walled Structures* 54(0):
330 140-155.

331 Dimopoulos C and Gantes C (2013) Comparison of stiffening types of the cutout in tubular wind
332 turbine towers. *Journal of Constructional Steel Research* 83(0): 62-74.

333 Efthymiou E, Kaziolas DN and Baniotopoulos CC (2009) On the response of steel lattice
334 telecommunication masts under environmental actions and seismic loading. In *Proceedings*
335 *of the Nordic Steel Construction Conference NSCC2009*. Malmo, Sweden, pp. 375-381.

336 EN 1993-1-1 (2005) Eurocode 3: Design of steel structures. Part 1-1: General rules and rules
337 for buildings. Brussels, Belgium. European Committee for Standardization (CEN).

338 EN 1993-1-6 (2007) Eurocode 3: Design of steel structures. Part 1-6: Strength and Stability of
339 Shell Structures. Brussels, Belgium. European Committee for Standardization (CEN).

340 Gencturk B, Attar A and Tort C (2012) Optimal design of lattice wind turbine towers. In
341 *Proceedings of the 15th World Conference on Earthquake Engineering*. Lisbon, Portugal, pp.
342 24-28.

343 Harikrishna P, Sanmugasundaram J, Gomathinayagam S and Lakshmanan N (1999) Analytical
344 and experimental studies on the gust response of a 52 m tall steel lattice tower under wind
345 loading. *Computers & structures* 70(2): 149-160.

346 Hibbitt, Karlsson and Sorensen Inc ABAQUS (2014) ABAQUS/Standard user's manual and
347 ABAQUS CAE manual. Version 6.12. Pawtucket, USA.

348 Holmes JD (1994) Along-wind response of lattice towers: part I-derivation of expressions for
 349 gust response factors. *Engineering Structures* 16(4): 287–292.

350 Holmes JD (1996) Along-wind response of lattice towers: part II-Aerodynamic damping and
 351 deflections. *Engineering Structures* 18(7): 483–488.

352 Holmes JD (1996) Along-wind response of lattice towers: part III-Effective load distributions.
 353 *Engineering Structures* 18(7): 489–494.

354 IEA (International Energy Agency) (2017) *United Kingdom renewable energy policy framework*
 355 *summary*. IEA, Paris, France. See
 356 <https://www.iea.org/policiesandmeasures/renewableenergy/?country=United%20Kingdom>
 357 (accessed 17/04/2018).

358 Khedr MAH (1998) *Seismic analysis of lattice towers*. (Diss). McGill University, Montreal,
 359 Canada.

360 Khedr MAH and McClure G (2000) A simplified method for seismic analysis of lattice
 361 telecommunication towers. *Canadian Journal of Civil Engineering* 27(3): 533-542.

362 Lavassas I, Nikolaidis G, Zervas P, Efthymiou E, Doudoumis I and Baniotopoulos CC (2003)
 363 Analysis and design of the prototype of a steel 1-MW wind turbine tower. *Engineering*
 364 *Structures* 25(8): 1097–1106.

365 Lee K and Bang H (2013) A study on the prediction of lateral buckling load for wind turbine
 366 tower structures. *International Journal of Engineering and Manufacturing* 13(10): 1829-1836.

367 Long H and Moe G (2012) Preliminary design of bottom-fixed lattice offshore wind turbine
 368 towers in the fatigue limit state by the frequency domain method. *Journal of Offshore*
 369 *Mechanics and Arctic Engineering* 134(3): 031902.

370 Long H, Moe G and Fischer T (2012) Lattice towers for bottom-fixed offshore wind turbines in
 371 the ultimate limit state: variation of some geometric parameters. *Journal of Offshore*
 372 *Mechanics and Arctic Engineering* 134(2): 021202.

373 Madugula MK (2001) *Dynamic response of lattice towers and guyed masts*. ASCE Publications,
 374 USA.

375 Nuta E, Christopoulos C and Parker JA (2011) Methodology for seismic risk assessment for
 376 tubular steel wind turbine towers: application to Canadian seismic environment. *Canadian*
 377 *Journal of Civil Engineering* 38(3): 293-304.

- Pineda I and Tandieu P (2016) *Wind in Power*. WindEurope, Brussels, Belgium.
- Rebelo C, Veljkovic M Simoes da Silva L, Simoes R and Henriques J (2012) Structural monitoring of a wind turbine steel tower – Part I: system description and calibration. *Wind Structures* 12(4): 285-299.
- Rebelo C, Veljkovic M Simoes da Silva L, Simoes R and Henriques J (2012) Structural monitoring of a wind turbine steel tower – Part II: monitoring results. *Wind Structures* 12(4): 301-311.
- Smith, M (2009) *Plasticity in ductile materials. Getting Started with ABAQUS/Explicit:Keywords Version, Version 6.14*. Simulia, Providence, RI.
- Stavridou N, Efthymiou E, Gerasimidis S and Baniotopoulos CC (2015) Investigation of stiffening scheme effectiveness towards buckling stability enhancement in tubular steel wind turbine towers. *Steel and Composite Structures* 19(5): 1115-1144.
- Stavridou N, Efthymiou E, Gerasimidis S and Baniotopoulos CC (2013) Modelling of the structural response of wind energy towers stiffened by internal rings. In *Proceedings of the 10th HSTAM International Congress on Mechanics*. Chania, Greece, pp. 190.
- Teng J and Rotter J (2004) *Buckling of Thin Metal Shells*. Taylor and Francis, London, UK.
- Timoshenko S and Gere J (1961) *Theory of elastic stability*. Tata McGraw-Hill Education, India.
- Tsitlakidou A, Fanouli L, Koltsakis E and Baniotopoulos CC (2005) Wind and ice design of ground based steel lattice masts. In *Proceedings of the 5th National Conference on Metal Structures*. Xanthi, Greece, pp. 207-214.
- Veljkovic M, Heistermann C, Husson W, Limam M, Feldmann M, Naumes J, Pak D, Faber T, Klose M, Fruhner KU, Krutschinna L, Baniotopoulos C, Lavassas I, Pontes A, Ribeiro E, Hadden M, Sousa R, Da Silva L, Rebelo C, Simoes R, Henriques J, Matos R, Nuutinen J and Kinnunen H (2009) *High-strength tower in steel for wind turbines (HISTWIN)*. European Commission, Brussels, Belgium, Report RFSR-CT-2006-00031.
- Zwick D, Muskulus M and Moe G (2012) Iterative optimization approach for the design of full-height lattice towers for offshore wind turbines. *Energy Procedia* 24: 297-304.
- Wolfram S (1999) *The mathematica*. Cambridge: Cambridge university press.

Figure captions

408	Figure 1. Tubular tower configuration and part lengths (in m) (a), shell thickness (in mm) (b) and
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431	Figure 21. Variation of tower weight with the increase of the top width for $\mu=1.2$
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434	
435	Table captions
436	Table 1. Tubular tower configuration (a), shell thickness (b) and mesh (c)
437	Table 2. Lattice tower subparts heights
438	Table 3. Lattice tower members' cross section properties

(a)

14
12
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14
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(b)

(c)

True stress - True strain curve

ODB: Eigenvalue.odb Abaqus/Standard 6.12-1 Mon Apr 09 09:05:03 GMT+03:00 2018

Step: Buckling
Mode 5: EigenValue = 2.2348

Deformed Var: U Deformation Scale Factor: +1.000e+00

S, Mises
SNEG, (fraction = -1.0)
(Avg: 75%)

Stress Level around the lower semi-circumference [points P1 and P14-P24]

Stress Level around the lower semi-circumference [points P2 - P13]

Tower weight over μ

Wtub
 Wlat min
 W350
 W400
 W450
 W500

Tower weight over μ

W_{tub}
 $W_{lat\ min}$
 W_{600}
 W_{700}
 W_{800}
 W_{900}
* W_{1000}

Tower weight over top width

— Wtub - - - Wlat min —*— W0.6

Tower weight over top width

— Wtub - - - Wlat min * W0.65

Tower weight over top width

— Wtub - - - Wlat min * — W0.7

Tower weight over top width

— Wtub - - - Wlat min —*— W0.75

Tower weight over top width

— W_{tub} - - $W_{lat\ min}$ * $W_{0.8}$

Tower weight over top width

Tower weight over top width

— Wtub - - - Wlat min *— W1.0

Tower weight over top width

— Wtub - - - Wlat min * — W1.05

Tower weight over top width

— Wtub - - - Wlat min —*— W1.1

Tower weight over top width

— Wtub - - - Wlat min *— W1.15

Tower weight over top width

Tower weight over top width

Tower weight over top width

Table 1

Table 1. Eigenvalues of tubular tower

	1 st Eigenvalue	2 nd Eigenvalue	3 rd Eigenvalue
Tubular Tower	2.0056	2.0229	2.2348

Table 2

Table 2. Lattice tower subpart heights

Tower subpart heights (m)					
Height	P-1	P-2	P-3	P-4	P-5
Subpart	34.45	55.53	68.19	75.64	76.15
Height					

Table 3

Table 3. Lattice tower cross sections

Diameter and thickness of tower cross-sections (mm)					
Legs	P-1	P-2	P-3	P-4	P-5
Diameter	411	371	352	340	286
Thickness	8	8	7	7	5
V-Brace					
Diagonals	P-1	P-2	P-3	P-4	P-5
Diameter	413	385	375	363	253
Thickness	7	7	7	7	5
V-Brace					
Horizontal	P-1	P-2	P-3	P-4	P-5
s	P-1	P-2	P-3	P-4	P-5
Diameter	342	282	240	216	214
Thickness	6	5	5	4	4