INTERNAL STIFFENING FOR STABILITY ENHANCEMENT OF STEEL WIND TURBINE TOWERS: NUMERICAL APPROACH AND COMPARATIVE STUDY

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

Institute of Metal Structures, Department Of Civil Engineering, Aristotle University of Thessaloniki
54124 Thessaloniki, Greece
e-mail: nstavrid@civil.auth.gr, vefth@civil.auth.gr

[1]Columbia University, Department of Civil Engineering and Engineering Mechanics
500 West 120th Street, New York, NY 10027, U.S.A.
e-mail:sg2988@columbia.edu

[2]School of Civil Engineering, University of Birmingham
B15 255 Birmingham, United Kingdom
e-mail: c.baniotopoulos@bham.ac.uk

SUMMARY

Wind energy, being probably the most promising renewable energy source due to its great energy potential and applicability, concluded recently to a variety of impressive relevant structural applications. As the most common type to support wind energy converters is the cylindrical steel tower, research on the structural optimization of the aforementioned shell structures is of great importance due to their high manufacturing and erection costs. In order to increase the wind energy harvesting, the improvement of the structural detailing of steel wind turbine towers is critical towards achieving economy in material use and structural robustness. The present paper addresses the stability performance of steel wind energy tubular towers and examines the introduction of internal stiffening rings along their height to reduce the possibility of appearance of local buckling phenomena and ultimately to decrease shell thickness. Aiming to contribute to better understanding of their structural behavior, the research work focuses on the development of reliable numerical models that predict accurately the aforementioned structural response by conducting a comparative study between different simulation approaches involving beam and shell modelling. Within the framework of the present study, finite element analysis is applied and the beneficial role of the aforementioned technique towards tower's structural response improvement is highlighted.

1. INTRODUCTION

Alternative energy production is considered crucial in order to improve the current climate conditions and limit the use of fossil fuels that are finite and produce harmful emissions. Wind energy due to its infinite nature and due to its remarkable development the past decades is a rather promising renewable energy source and therefore research towards that direction is beneficial to the overall energy production. The optimized design of a wind turbine tower, being the basic structural part of a wind converter is of great importance in order to achieve more robust structures and more economic design. The wind turbine towers that are used more fre quently and vary in structural configuration are: a cylindrical tower, a jacket tower with a truss structure and a hybrid tower meaning a combination of a truss structure for the lower part and a tube part for the upper one.

The most commonly applied wind turbine tower type is a steel cylindrical shell tower due to its structural detailing, easier mounting and limited labour done on site since the tower parts are produced in the factory and are only mounted on site. The tower is composed of subsequent cylindrical or conical parts that are transported and mounted on site [1].The structural analysis of the main supporting structure of wind generators is considered of high importance since failure in such projects has great economical, structural and safety losses. The governing loads acting on the tower are the wind pressure up the tower height, the moment and the lateral load due to the rotor's function and the vertical load that is equal to the rotor weight. In the already constructed wind farms there has been a number of accidents attributed to structural failure and more specifically to buckling phenomena [2], [3], therefore it is crucial to investigate the nature of these problems and to provide structural solutions. Wind turbine towers are simple tubular cantilever structures, that due to their geometry can carry great loads with small shell thickness. The investigation of the buckling behavior of cylindrical shells in general has been part of the research work, both numerical and experimental, conducted in the past by Timoshenko and Gere [4], Bazant and Cedolin[5], Teng and Rotter [6]. Tubular steel wind turbine towers lie in the field of cylindrical shells under combined loading and special research work has been devoted to the behavioral analysis of those structures and the explanation of their main structural problems. A prototype 1 MW steel wind turbine tower has been analyzed and designed by Lavassas et al. [7] and a stability analysis of a steel wind turbine tower was conducted by Bazeos et al.[1]. Lee and Bang [8] performed a numerical comparison between the simulation and a real collapsed wind turbine tower, while Dimopoulos and Gantes [9] have provided experimental and numerical results on different types of stiffening around the openings of wind turbine towers. Arasu et al [10] and Nuta et al [11] have performed seismic analyses of wind turbine towers. Despite the scientific research carried out in the field of cylindrical shells and their structural behavior, limited work has been devoted to the numerical analysis of wind turbine towers in specific. In order to better understand and explain the response of these tall and slender structures against the complex loading conditions they have to sustain, numerical analysis is performed. Aiming to contribute to better understanding of their structural behavior, the research work focuses on the development of reliable numerical models that predict accurately the aforementioned structural response by conducting a comparative study between different simulation approaches involving beam and shell modeling. The numerical analysis with beam modeling and shell modeling are performed with the use of commercial finite element software Simulia [12]. The present paper addresses the stability performance of steel wind energy tubular towers and examines the introduction of internal stiffening rings in the shell model in order to reduce the possibility of appearance of local buckling phenomena and ultimately to decrease shell thickness.

2. BEAM MODELING

The wind turbine tower that is used for the comparative study of the numerical modelling of wind turbine towers has a total length of 76.15 meters and consists of 3 parts that are assembled on site due to transportation limitation of longer elements. For the first approach the tower is modelled with beam elements B31 as described in the Abaqus Manual [12]. The tower is simulated with beam elements and therefore the reference axis is the one passing from the center of gravity of the cylinder's cross section. The whole tower is divided into parts with varying wall thickness. Since the software does not provide beam elements with conical cross sections, the tower is divided into cylindrical parts, small enough so that the conical cross-section is better approached. The beam element used has both translational and rotational degrees of freedom in its nodes. When modelling a wind turbine tower with beam elements 3-dimensional real tower part is simulated by an 1-dimentional beam element. In comparison, for the simulation with shell elements, the 3-dimensional tower part is modelled with a 2-dimensional shell at the reference level where the stresses are calculated and the thickness is set by the user.

The original constructed tower is divided in 3 different parts of lengths 21.8 m, 26.6 m and 27.8 m from bottom to top. The lower diameter of the tower is 4,3 m and the top one is 3 m. The thickness of the shell wall is not constant, ranging from 30mm at the bottom to 12mm at the top. The model has been constructed by subsequent linear elements that have different shell thickness and diameter.

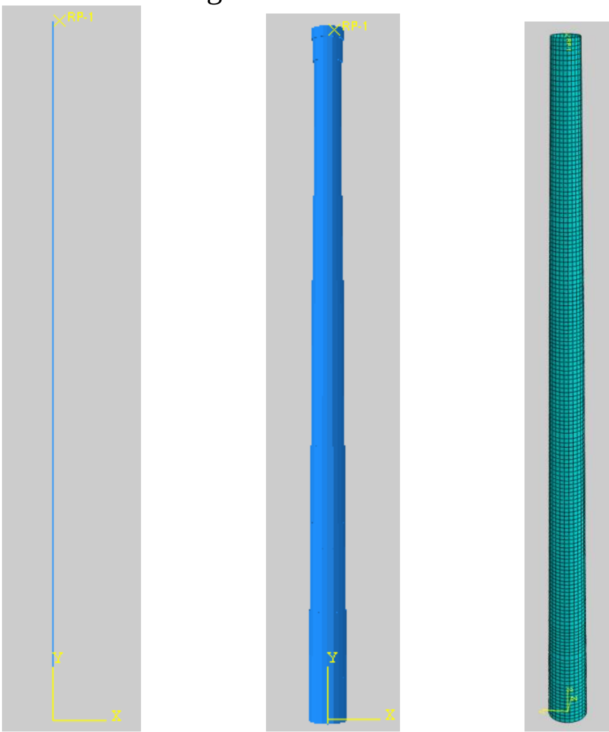
In the numerical model, the concentrated loads are applied at the top of the tower to a reference point eccentrically set to the middle axis of the tower, simulating the exact position of the rotor. The reference point is external to the top beam element and is coupled to the tower top with the appropriate constraint. The gravity loads are automatically calculated through the density of the material and the wind loading is taken into account as a distributed load along the tower height. The loads acting on the wind turbine tower are given in equation (1) and are: the vertical loading due to the nacelle weight (V), the horizontal loading (H) and moment (M) due to the rotor function and the distributed wind loading (W) on the tower shell.

$$P = \{V + H + M + W\} \qquad (1)$$

A material nonlinear analysis is performed to examine the tower response towards this combined loading. The material data used in a non-linear analysis for steel S355 are the following: Poisson's coefficient 0.3, Young's modulus 210GPa and for steel class S355 the yield stress is considered 350MPa and ultimate strength 510MPa. In order to introduce plasticity data, the material properties have to be considered in terms of plastic true stress and plastic true strain.

3. SHELL MODELING

The constructed wind turbine tower structure under investigation is the same as the one modelled with beam elements as shown in figure 1. In shell modelling it is modeled with shell elements of type S4R as described in the Abaqus Manual [12]. The conventional shell model geometry is specified at the reference surface which is considered to be the middle surface and the thickness of the shell is defined by the section properties. The shell element used has both translational and rotational degrees of freedom in its nodes. The difference between shell and continuum elements is that in the latter the full 3-D geometry is specified and therefore, the element thickness is defined by the nodal geometry and the elements provide only translational degrees of freedom in the nodes.



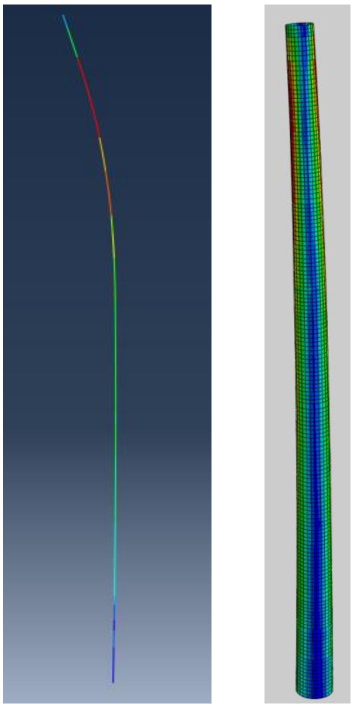
(a) (b) (c)

Fig.1 Example of the different modelling techniques. Beam modelling(a), Beam modelling rendered (b) and shell modelling (c).

The tower is again modelled into three consequent parts. In the numerical model, the concentrated loads are applied at the top of the tower to a reference point taking into account the eccentricity. The gravity loads are automatically calculated through the density of the material and the geometry of the shells, while the wind loading is taken into account as pressure on the outer circumference of the shell. The pressure is not uniform due to the fact that each point on the circumference has a different pressure due to the different angle towards the wind direction. In order to compare the beam simulation and shell simulation, Material Non-linear Analysis is applied. As observed in the results of the present study the difference between the two different types of modeling is not great. The difference is observed when performing GMNIA. In order to obtain the buckling shapes of the structure and the eigenvalues being necessary for the non-linear static imperfection analysis, a buckling analysis is first performed. The GMNIA is applied only on the tower modelled with shell elements, since no such imperfections can be calculated with beam models. In the shell structure internal stiffeners are added in order to observe their applicability and influence on the buckling shapes and ultimate capacity of the tower. A comparison of the buckling shapes of the stiffened and the unstiffened structure can be made from this step along with a comparison of the critical buckling load of the two structures. Performing an eigenvalue analysis only with the wind loading is also important to observe the beneficial impact of the implementation of stiffening rings towards wind loading. The material data used in a non-linear analysis for steel S355 has the same characteristics with the one used in the beam modelling.

4. COMPARATIVE RESULTS

In both modeling techniques the tower was analyzed performing material nonlinear analysis (MNA). The results when not having varying distribution of wind loading depending on the angle towards the wind, were very similar between shell modelling and beam modelling as shown in figure 2. Failure is concentrated at the top part of the tower in both cases. In the beam model section, thin shell is selected in order for equations of thin-walled theory to be taken into account. However, since it is a linear model and beam theory assumptions are made, no clear view of the location of the deformation can be extracted. In the shell model valuable information of the exact location of the deformation are extracted and different positions around the circumference of the shell have different stresses and strains, while in the beam modelling all the circumference is represented by one point taking an average value which is rather coarse. It is rather important to observe the exact position of local buckles and deformations since shell structures like wind turbine towers are proved to be very sensitive to initial imperfections and local failures can lead to total collapse of the structures.



(a) (b)

Fig.2 MNA results for the beam model (a)and shell model (b).

5. INFLUENCE OF STIFFENING RINGS

The tall and slender shell structures like wind turbine towers are proved numerically and experimentally vulnerable to initial imperfections, which are the main reason leading to local buckling and progressively to the overall collapse of the structure. By modeling the tower with beam elements the overall circumference at each level is represented by one point only which cannot describe the response of the tower in detail. As we can see from the comparative study, both modeling techniques indicate the upper part of the tower as the more probable to be led to material failure. Even the beam modeling can describe the tower response and predict material failure. The advantage that the shell modeling has is that wind loading can be better modeled with varying values around the circumference of the shell, depending on the position towards the wind direction. The loading condition of the tower is given in equation (1). The tower, as a shell structure is very sensitive to initial imperfections and boundary conditions. It is characteristic that the ultimate capacity of the tower against the combined loading situation is almost 30% lower when applying initial imperfections to the analysis as observed in Table 2. Since the influence of the initial imperfections is that great it is judged important to include them in the analysis of wind turbine towers as also indicated in the European Standard for shell structures [13]. A solution to secure wind turbine towers against wind loading and also against imperfections, is the implementation of internal stiffening rings. The function of those stiffeners is working mainly against the varying external pressure due to wind loading, which can be observed by the great difference in the first eigenvalue of the stiffened and unstiffened structure shown in Table 1.

	1st Eigenvalue with combined loading	1st Eigenvalue with wind loading
Tower without internal rings	2,0056	27118
Tower with 21 internal rings	2,3832	37258
Difference	**18,82%**	**37,40%**

Table 1. Eigenvalue Analyses of stiffened and unstiffened shell structure

	MNA	GMNIA	Difference MNA	Difference GMNIA
Tower without internal rings	0,61 P	0,44 P	-	-
Tower with 21 internal rings	0,68 P	0,54 P	**11,48%**	**22,72%**

Table 2. MNA and GMNIA results of stiffened and unstiffened shell structure

The main effect of wind loading as observed in Figure 3, is the tendency to ovalize the circular cross-section of the tower imposing high circumferential stresses to the shell. This distortion of the shell circumference can be prevented with the introduction of internal stiffening rings. Moreover, the generally limited buckling strength of the shell on the circumferential direction due to the small thickness-to-radius ratio of the tower shell is increased with the introduction of the circular stiffeners as observed in Table 2. Finally, the modeling concept one should have is when investigating the behavior of wind turbine towers is that the main failure mode is buckling and not material failure. The classical beam elements that simulate the whole circumference with one point cannot predict local buckling failure that can be predicted with shell modeling. As it is pointed out from Figure 4, nonlinear analysis with shell elements (US_MNA) stops when buckling failure occurs since it cannot provide solution for loss of stability. The structures when modeled with beam elements appear stiffer, due to the beam-theory assumptions and due to the fact that wind loading is modeled only with a distributed loading up the height of the tower without taking into account the varying distribution around the circumference. The curve diverges from the straight line due to material failure and local buckling phenomena are not taken into account. For the shell modeling the fact that the analysis stops, shows that loss of stability occurs. In future work other algorithms appropriate for these cases will be used to examine the post-buckling behavior of theses structures.

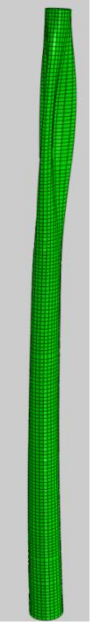


Fig.3 1st Eigenmode of unstiffened structure against wind loading

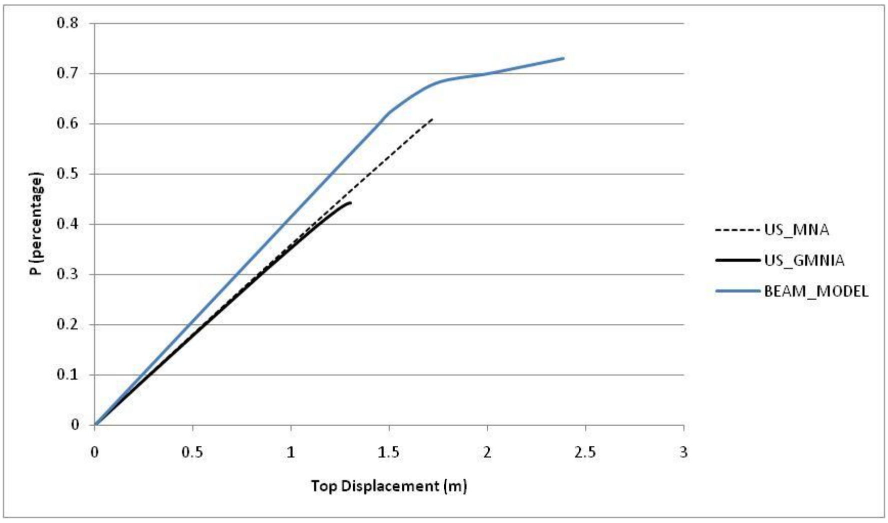


Fig.4 Load – Top displacement curves for unstiffened structure

6. CONCLUSIONS

The beam element modeling compared to the shell element modeling provides limited data and does provide tools for the examination of local buckling phenomena. The difference in the tower response with the use and the absence of initial imperfections shows the crucial role that they play to the overall response of the tower. Cylindrical shells have been proved to buckle in experiments in load levels much lower than the theoretical calculated value and this behavior is attributed to the presence of initial imperfections that are not taken into account in beam models or shell models with the assumption of a "perfect" structure. Second order phenomena are proved to be very important in interpreting the behavior of wind turbine towers. The solution of implementation of internal stiffening rings is proved to have a beneficial impact against the pressure due to wind loading, thus limiting the ovalization of the tower's cross-section and leading the structure to have a higher buckling load compared to the unstiffened structure.

7. REFERENCES

[1] Bazeos N., Hatzigeorgiou G., Hondros I., Karamaneas H., Karabalis D.L., and Beskos D. "Static, seismic and stability analyses of a prototype wind turbine steel tower", Engineering Structures, Vol. 24, No. 8, 2002, pp. 1015-1025.

[2] Ragheb M. "Safety of Wind Systems", University of Illinois Course Textbook, 2012.

[3] Kang-Su L. and Hyung-Joon B. "A Study on the Prediction of Lateral Buckling Load for Wind Turbine Tower Structures", International Journal of Precision Engineering and Manufacturing, Vol. 13, 2012, pp. 1829-1836.

[4] Timoshenko S. and Gere J. "Theory of elastic stability", Tata McGraw-Hill Education, 1961

[5] Bazant Z. and Cedolin L. "Stability of Structures. Elastic, Inelasti, Fracture and Damage Theories, World Scientific, 2010.

[6] Teng J. and Rotter J. "Buckling of Thin Metal Shells", Taylor and Francis, 2004

[7] Lavassas, I., Nikolaidis G., Zervas P., Efthimiou E., Doudoumis I. and Baniotopoulos C.C. "Analysis and design of the prototype of a steel 1-MW wind turbine tower", Engineering Structures, Vol. 25, No. 8, 2003, pp. 1097-1106.

[8] Lee K. and Bang H. "A study on the prediction of lateral buckling load for wind turbine tower structures", International Journal of Precision Engineering and Manufacturing, Vol. 13, No. 10, 2013, pp. 1829-1836.

[9] Dimopoulos C., Gantes C. "Experimental investigation of buckling of wind turbine tower cylindrical shells with opening and stiffening under bending", Thin-Walled Structures, Vol. 54, 2012, pp. 140-155.

[10] Arasu P., Sagayaraj D. and Gowrishankar J. "Seismic analysis of a wind turbine steel tower", Hyper Works Technology Conference, 2011.

[11] Nuta E., Christopoulos C. and Packer J. "Methodology for seismic risk assessment for tubular steel wind turbine towers: application to Canadian seismic environment", Canadian Journal of Civil Engineering, Vol. 38, No. 3, 2011, pp.293-304.

[12] Dassault Systemes Simulia Corp. "Abaqus Documentation", 2012.

[13] CEN, "Design of Steel Structures-General Strength and Stability of Shell Structures", EN 1993-01-06, 2006.

ACKNOWLEDGEMENTS

The first author wants to express her gratitude to IKY for financial support of the research activities on performing her PhD thesis through the IKY Fellowship of Excellence for Postgraduate Studies in Greece – SIEMENS Program

ΔΙΑΣΦΑΛΙΣΗ ΤΗΣ ΕΥΣΤΑΘΕΙΑΣ ΧΑΛΥΒΔΙΝΩΝ ΠΥΡΓΩΝ ΑΝΕΜΟΓΕΝΝΗΤΡΙΩΝ ΜΕΣΩ ΕΣΩΤΕΡΙΚΗΣ ΕΝΙΣΧΥΣΗΣ. ΑΡΙΘΜΗΤΙΚΗ ΚΑΙ ΠΑΡΑΜΕΤΡΙΚΗ ΑΝΑΛΥΣΗ

Ναυσικά Σταυρίδου, Ευάγγελος Ευθυμίου, Σίμος Γερασιμίδης[1], Χαράλαμπος Κ. Μπανιωτόπουλος[2]

Αριστοτέλειο Πανεπιστήμιο Θεσσαλονίκης, Πολυτεχνική Σχολή,
Τμήμα Πολιτικών Μηχανικών, Εργαστήριο Μεταλλικών Κατασκευών

e-mail:nstavrid@civil.auth.gr, vefth@civil.auth.gr

[1]Columbia University, Department of Civil Engineering and Engineering Mechanics
500 West 120th Street, New York, NY 10027, U.S.A.
e-mail:sg2988@columbia.edu

[2]School of Civil Engineering, University of Birmingham
B15 2TT Birmingham, United Kingdom
e-mail:c.baniotopoulos@bham.ac.uk

ΠΕΡΙΛΗΨΗ

Η αιολική ενέργεια, πιθανότατα αποτελεί τον πλέον ελπιδοφόρο τομέα ανανεώσιμων πηγών ενέργειας λόγω του μεγάλου δυναμικού και της ευρείας εφαρμογής της, οδηγώντας πρόσφατα σε πληθώρα εντυπωσιακών κατασκευών παραγωγής της. Καθώς ο πιο συνήθης τύπος ανεμογεννήτριας είναι ένας κυλινδρικός χαλύβδινος πύργος, η έρευνα για τη δομική βελτιστοποίηση των κελυφωτών αυτών κατασκευών είναι υψηλής σημασίας εξαιτίας του υψηλού κόστους παραγωγής και ανέγερσής τους. Για να γίνει καλύτερη εκμετάλλευση του αιολικού δυναμικού, η βελτίωση του δομικού συστήματος των πύργων ανεμογεννητριών είναι σημαντική προς την επίτευξη πιο οικονομικού σχεδιασμού, οικονομίας υλικού και ανθεκτικότητας της κατασκευής. Η παρούσα εργασία επικεντρώνεται στην ευστάθεια των χαλύβδινων κυλινδρικών πύργων ανεμογεννητριών και εξετάζει την εισαγωγή εσωτερικών δακτυλίων ενίσχυσης του κελύφους με στόχο τη μείωση της πιθανότητας εμφάνισης τοπικού λυγισμού και τελικά της μείωσης του πάχους του κελύφους. Για την καλύτερη κατανόηση της δομικής συμπεριφοράς των πύργων, η εργασία επικεντρώνεται στην ανάπτυξη αξιόπιστων αριθμητικών προσομοιωμάτων ώστε να προβλεφθεί με ακρίβεια η απόκριση του φορέα. Η μεθοδολογία ανάπτυξης του αριθμητικού προσομοιώματος περιλαμβάνει τη διενέργεια συγκριτικής ανάλυσης αριθμητικών προσομοιωμάτων με τη χρήση πεπερασμένων στοιχείων κελυφών και στοιχείων δοκού. Στο πλαίσιο της παρούσας εργασίας, γίνεται εφαρμογή της ανάλυσης πεπερασμένων στοιχείων και καταδεικνύεται ο ευεργετικός ρόλος της ενίσχυσης με εσωτερικούς δακτυλίους στη συνολική δομική συμπεριφορά των πύργων.