COMPARATIVE STUDY OF STIFFENING CONFIGURATIONS AT THE DOOR OPENING OF A 2 MW WIND TURBINE TOWER

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1. ABSTRACT

This paper presents a comparative study of the most used stiffening configurations for the door opening of a 2 MW wind turbine tower. Seven configurations are compared.
The comparative study is performed using (LBA) and (GMNA) analyses on the perfect shell for the evaluation of the limit load for the entire tower. The design method used is through "Global numerical analysis" ([3] §8.6).
The analyses are performed on an overall model having dense 2D and 3D FE mesh, that describes the whole tower with all its structural details included (Door opening & stiffenings, connection flanges, foundation, anchoring details), as well as with the use of local FE models describing only the part of the tower at the door opening, in which the tower section forces are calculated using a simpler linear model.
As resulted from the paper, the most efficient stiffening method is the use of a very thick ring around the door opening, while the stiffeners to the tower shell around the door are not providing satisfactory results. The use of local models is satisfactory (93-95%) approaching the stress state at this area in comparison to the overall model.

2. INTRODUCTION

The prototype tower examined corresponds to a 2 MW wind turbine. The height of the tower is 76.15 m, and the total height of the wind turbine including the rotor and the blades is 123 m. The shell diameter at the base is 4.30 m and the diameter at the tower top is 3.0 m. Shell thicknesses vary from 30 mm at the bottom to 12 mm at the top. The tower is divided into three parts connected together by bolted flanges. The steel quality is S355 and the fabrication Class is B. It is worthy to note that the steel tower is embedded to the reinforced concrete foundation. For the analysis, a full FE model (Fig. 1) has been developed for the tower and the foundation with all the structure details included (flange connections, door opening, anchoring detail etc.). A linear model has been also developed for the cross-checking of the results of the aforementioned advanced FE model. Then, a variety of stiffening configurations has been applied to the door opening.

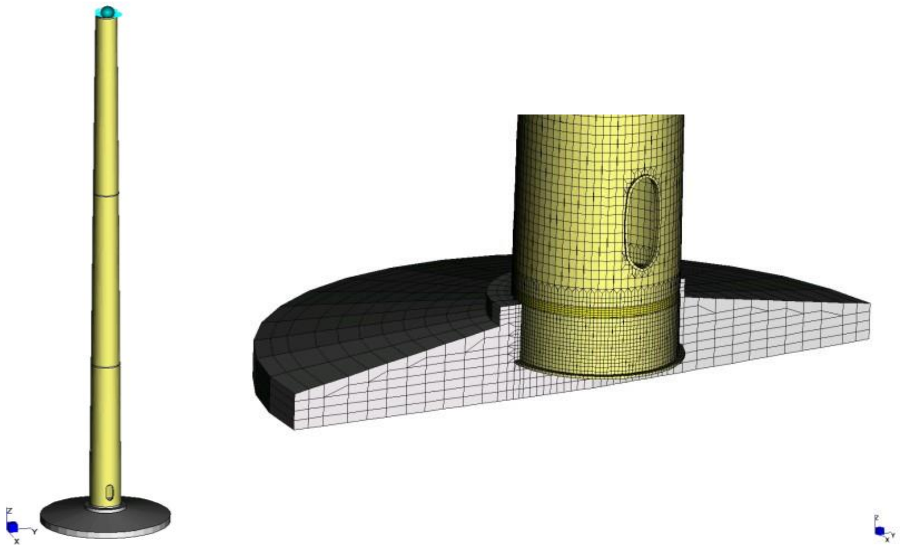

Fig. 1: FE model and detail at base position

The door opening induces a significant dissipation of the shell stresses and at the same time an inevitable magnification of their magnitude, for the meridional (σ_{zz}) component. It must be noted that the later is the critical stress at the vicinity of the door opening [9]. The ultimate limit states under examination in the present text are the plastic limit state [LS1] and the buckling limit state [LS3]. The main objective of stiffening the area around the door is to:

- Control the local stresses, in order to prevent the excessive loading of the relevant shell courses.
- Provide adequate lateral support to the shell and establish thus adequate resistance against local buckling.

3. STIFFENING CONFIGURATIONS

Seven stiffening configurations around the opening have been tested (Fig. 2 & Table 1).

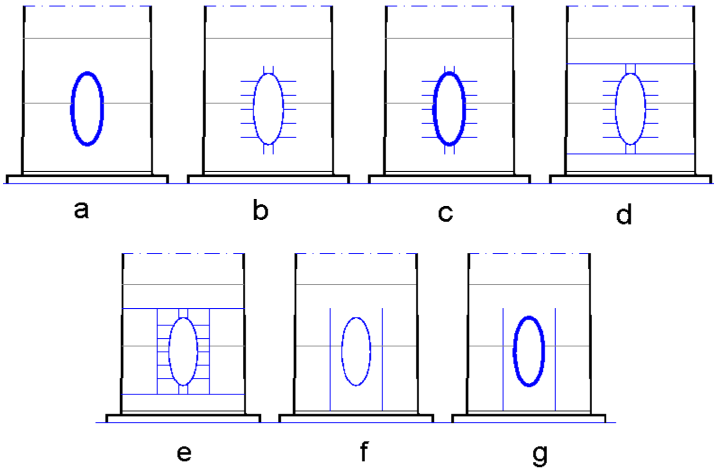

Fig. 2: Stiffening configurations

Type	[a]	[b]	[c]	[d]	[e]	[f]	[g]
Ring thickness (mm)	70	30	60	30	30	30	70
Stiffeners vertical to the ring (mm)		30	30	30	30		
Rings to the shell below & above the door (mm)				30	30		
Vertical stiffeners at both sides of the door (mm)					30	30	30

Table 1: Stiffening configurations

4. ANALYSIS AND DESIGN

A Linear Buckling Analysis (LBA) and an analysis for the Plastic Limit Load of the tower has been performed to all cases. In the LBA, the first 10 eigenvalues appear in various

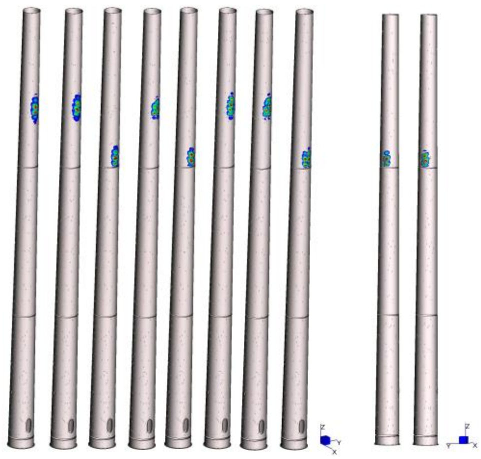

Fig.3: Eigenmodes: Positive 1-5,7,10 & Negative 6,9 (neglected)

positions in the top part of the tower, where the shell thicknesses are smaller (Fig. 3). From the LBA results, one may conclude that the tower is about to reach its Ultimate Load through buckling in one of those positions. But this type of analysis assuming the tower as linear elastic is not taking into account that simultaneous plasticization may trigger shell buckling in another position.

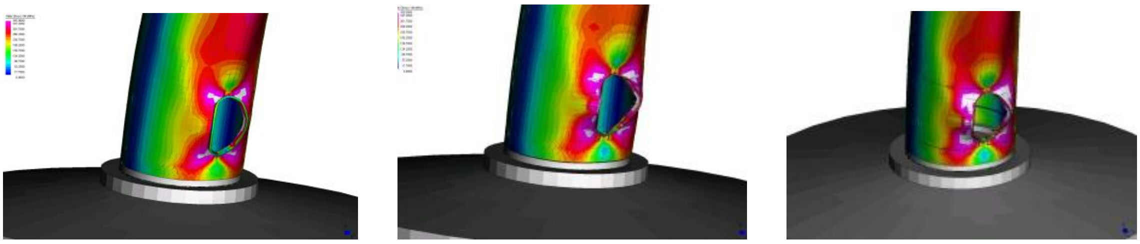

Fig. 4: Types [a],[b],[c]

Plastic Limit Load analysis is performed by increasing the wind load step by step until the total failure of the structure. This can be checked by monitoring the rotation or displacement at various critical checkpoints. In this analysis, in all cases, the tower reaches its ultimate load through local buckling at the position of the door, which happens because of the local plasticization and the drastic reduction of the Elasticity modulus in this area. Table 2 presents a comparison of the global limit factors for all cases, and the corresponding resistance ratios for the tower.

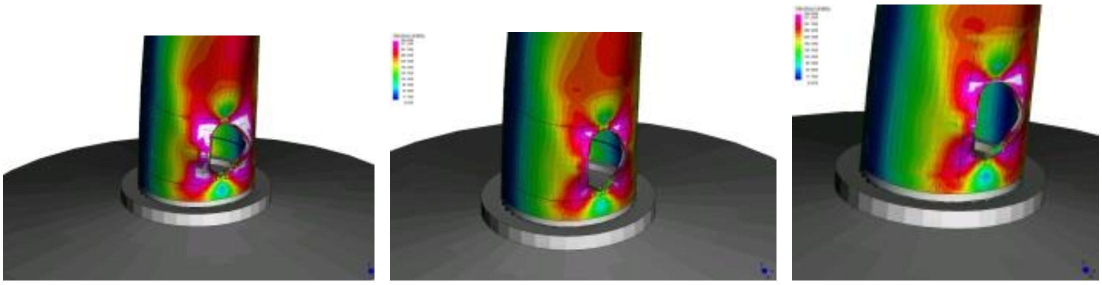

Fig. 5: Types [d],[e],[f]

Determining the analysis results, all types of stiffening arrangements examined seem to be satisfactory and are adequate in preventing the buckling of the shell around the door opening. A rigid ring around the door opening is always mandatory, even if additional

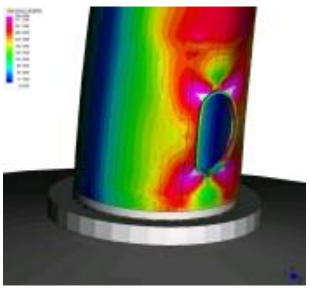

Fig. 6: Type [g]

stiffening plates are installed. Horizontal stiffeners around the door have not significant impact to the resistance of the opening. Vertical stiffeners are more effective, since they undertake the meridional stresses. On the other hand, in all cases where a thin ring is put around the door regardless of the type of stiffeners that are put to the shell, the ring is running plasticized, even before step by step analysis reaches the extreme wind (G+1.50W) combination value. The governing stress in all cases is the meridional (σ_{zz}) and the vertical segment of the ring accumulates the major part of the stresses that otherwise would pass through the opening area (Figs. 4-6).

Type	r_{Rk}	r_{Rd}
[a]	2.05	1.45
[b]	1.95	1.38
[c]	2.05	1.45
[d]	1.95	1.38
[e]	2.00	1.42
[f]	1.95	1.38
[g]	2.10	1.49

Table 2: Comparison of the various stiffening configurations

5. ANALYSIS WITH THE AIDE OF PARTIAL FE MODELS

Two additional models have been developed, incorporating the lower part of the overall model, the scope of which is the check of the accuracy of the analyses, when the calculations are performed by hand or by the use of a linear model. These models are:

- [A]: The model comprises the first two bottom courses, and the foundation.
- [B]: The model comprises the first two bottom courses, clamped to the base. The foundation is not implemented.

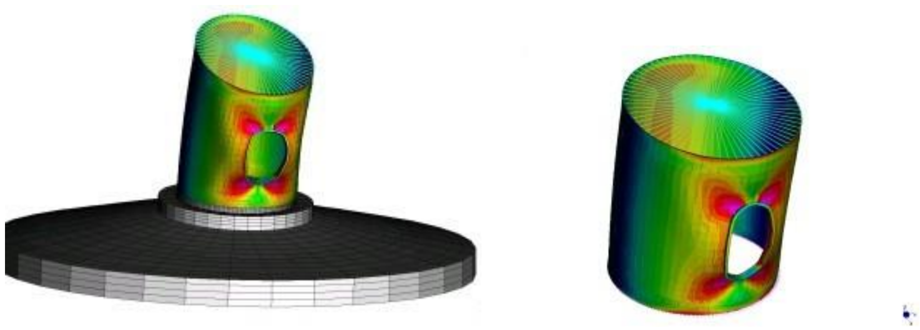

Fig. 3: Partial models [A] and [B]

This procedures require less modeling and computational effort. But as the section forces (M_x,M_y,V_x,V_y and N) derived by hand calculation or from the linear model, are applied to the top, along the free boundary circumference, with the following assumptions:

- The influence of the circumferential stresses to the shell, induced by the specific wind load distribution on the tower stem has been ignored [9].
- In linear model calculations, the Euler-Bernoulli assumption is directly adopted. With the objective of realizing this assumption to the partial FE model, all the nodes of the force application level are connected to each other by means of special rigid links. At the same time, the implementation of this technique allows the smooth transfer of the sectional forces to the shell.

As evidenced by the analyses results in Table 3, the convergence between the stress state of the global and the partial models can be considered satisfactory in general, the Von Mises stresses derived from the partial models being no more than 7% lower.

VM stresses (MPa)		
Model	Shell	Ring
Global	340	348
Partial [A]	331	334
Partial [B]	320	325

Table 3: Stress comparison for the partial models (Extreme wind comb.)

6. CONCLUSIONS

In all types of stiffening arrangements that have been tested, the ultimate load for the tower is located at the vicinity of the door opening.

As results from the above analyses, the presence of a rigid ring around the door provides the best reinforcement to the opening, in contradiction to the other types of stiffeners, the contribution of which is comparatively less effective.

Stiffening by means of horizontal stiffeners around the door has not significant impact to the resistance of the opening. Vertical stiffeners are more effective, since they efficiently undertake the meridional stresses. Even to the door ring, it is the vertical segment which is fully stressed.

For the specific tower, the use of a linear model or a hand-calculation approach, along with an additional FE model for the door detailing, results to a 5% ÷ 7% decreased stress state, compared to the one corresponding to the more accurate full model.

7. ACKNOWLEDGEMENTS

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8. REFERENCES

[1] EN 1993-1-4: "Actions on structures – General actions", 2005.

[2] EN 1993-1-1: "Design of steel structures – General rules and rules for buildings", 2005.

[3] EN 1993-1-6 "Design of steel structures – General strength and stability of steel structures", 2007.

[4] EN 1998-1 "Design of structures for earthquake resistance – General rules, seismic actions and rules for buildings", 2004.

[5] EN 1998-6 "Design of structures for earthquake resistance – Towers, masts and chimneys", 2005.

[6] GL Wind 2003 IV – Part 1: "Guideline for the Certification of Wind Turbines", 2004.

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

[8] Lavassas I., Nikolaidis G., Zervas P., C.C. Baniotopoulos "Design of large scale wind turbine towers in seismic areas", 7th National Conference on Steel Structures, Volos Greece, 2012, Vol. 1, pp. 272 – 278.

[9] Baniotopoulos, C.C., Lavassas I., Nikolaidis G., Zervas P., "Design of large scale wind turbine towers in seismic areas", STESSA 2012 Conference - Behaviour of Steel Structures in Seismic Areas, Santiago, Chile, CRC Press 2012, pp. 319 – 324.

[10] Baniotopoulos, C.C., Lavassas I., Nikolaidis G., Zervas P., "Stiffeners to the door opening of the tower", RFS-CT-2006-00031 - HISTWIN research programme. Work Package 3.4, 2009.

ΣΥΓΚΡΙΤΙΚΗ ΔΙΕΡΕΥΝΗΣΗ ΔΙΑΤΑΞΕΩΝ ΕΝΙΣΧΥΣΗΣ ΑΝΘΡΩΠΟΘΥΡΙΔΑΣ ΠΥΡΓΟΥ ΑΝΕΜΟΓΕΝΝΗΤΡΙΑΣ 2 MW

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ΠΕΡΙΛΗΨΗ

Στην παρούσα εργασία, γίνεται συγκριτική διερεύνηση των συχνότερα χρησιμοποιούμενων διατάξεων ενισχύσεως γύρω από την ανθρωποθυρίδα πύργου ανεμογεννήτριας 2 MW ύψους 76.15 μέτρων, και της επιρροής τους στην αντοχή του. Συγκρίνονται επτά διατάξεις ενισχύσεως. Η σημερινή συνήθως εφαρμοζόμενη πρακτική είναι η χρήση τοπικών μοντέλων πεπερασμένων στοιχείων, με γραμμική ανάλυση (LA) και "Σχεδιασμό μέσω τάσεων" (EN1993-1-6 παρ 8.5).

Στην εργασία αυτή, πραγματοποιείται συγκριτική μελέτη με αναλύσεις (LBA) και (GMNA) για την αποτίμηση της οριακής αντοχής του πύργου και τον σχεδιασμό του με χρήση "Καθολικής αριθμητικής ανάλυσης" (EN 1993 1-6 παρ. 8.6).

Οι αναλύσεις πραγματοποιούνται σε συνολικό μοντέλο με πυκνό δίκτυο επιφανειακών και χωρικών πεπερασμένων στοιχείων, που περιλαμβάνει τον πύργο με όλες τις δομικές του λεπτομέρειες (Ανθρωποθυρίδα με την ενίσχυση της, φλάντζες σύνδεσης, θεμελίωση, διάταξη αγκύρωσης), καθώς και με τοπικά μοντέλα που περιλαμβάνουν μόνον το τμήμα του πύργου στη θέση της ανθρωποθυρίδας, στα οποία η μεταφορά των εντατικών μεγεθών γίνεται από απλούστερο συνολικό μοντέλο του πύργου.

Από τα αποτελέσματα της εργασίας προκύπτει πως η πιο ενδεδειγμένη μέθοδος ενίσχυσης είναι η τοποθέτηση ενός δαχτυλιδιού με πολύ μεγάλο πάχος γύρω από την ανθρωποθυρίδα, ενώ οι ενισχύσεις του κελύφους με stiffeners δεν προσφέρουν ικανοποιητικά αποτελέσματα. Επίσης, η χρήση τοπικών μοντέλων προσεγγίζει με ικανοποιητική ακρίβεια (93-95%) την εντατική κατάσταση στη θέση της ανθρωποθυρίδας σε σύγκριση με το συνολικό μοντέλο.