Design of large scale wind turbine towers in seismic areas

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ABSTRACT: In the present paper, the design of the prototype of large tubular steel wind turbine tower in earthquake areas is presented. For the simulation of the tower's structural response, two models have been developed, a linear model and an overall Finite Element model in which all the tower details are included (flange connections, door opening, foundation, anchoring). The tower has been designed for plastic and buckling limit states, for wind loading and for seismic loading as well, using both calculation models according to the provisions of the relevant Eurocodes. A geometric, material and boundary condition non linear analysis (including global and local shell imperfections) of the tower is performed for the wind loading. In addition, a buckling analysis and a limit load analysis has been carried out for the whole structure. Finally, the three methods proposed by the Eurocodes for the local buckling design of the shell have been compared. Concerning the design against earthquake, an eigenvalue analysis along with a response spectrum analysis has been performed according to the Eurocode 8 specifications. The behavior of the tower for earthquake loading is compared to the corresponding for wind loading one regarding both computational models.

1 INTRODUCTION

The wind turbine tower at hand is macroscopically a cantilever whose section forms a thin-walled cylindrical shell. Therefore several issues arise during the analysis such as the local buckling of the shell structure or the stress concentrations around the door opening which must be thoroughly examined (Baniotopoulos et al. 2011). The prototype tower examined carries 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 interconnected by bolted flanges. The steel quality of the tower body is S355 and the fabrication Class is B (EN 1993-1-6). It is worthy to note that the steel tower is embedded into the reinforced concrete foundation.

2 MODELLING

2.1 *Model description*

For the analysis of the tower, a detailed FE model has been developed for the tower and the foundation with all the structural details included (flange connections, door opening, anchoring etc.). A linear model has been also developed for the cross-checking of the results of the aforementioned advanced FE model.

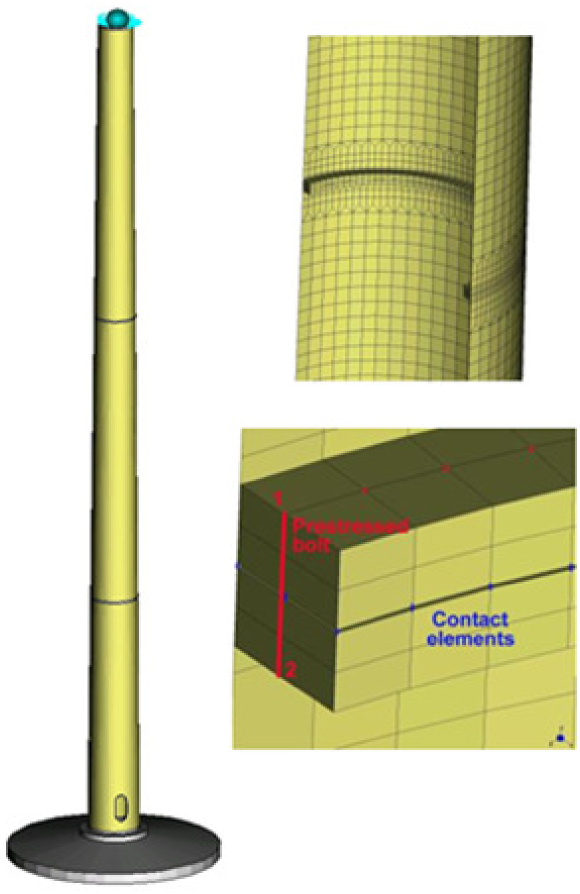

Figure 1. Wind turbine tower – FE model

The overall FE tower model (Figure 1) is composed by the shell, the intermediate flanges and the embedded to the foundation skirt. The reinforced concrete foundation is also modeled. The structural model becomes denser in the vicinity of the flanges, the door opening and the base ring in order to more accurately describe the local concentration of the stresses. For a more efficient use of the FE software, a cylindrical coordinate system is used. The shell is divided along the height into skirts of varying thicknesses, each of which constitutes an individual FE group. The element arrangement along the circumference is dictated by the number of bolts of the connection flanges. This way, there is a node at each bolt position.

The intermediate flanges (Figure 1) are modeled by the use of brick elements. The interfaces of the flanges are connected by means of frictional unilateral contact elements being active only in compression. On bolt positions, the upper node of the upper flange is attached to the lower node of the lower flange via the pre-stressed linear elements of cable type being active only in tension. Especially for the top flange, the contribution of the nacelle to the horizontal rigidness of the section is achieved by the introduction of master – slave links, converging at the center of the circle.

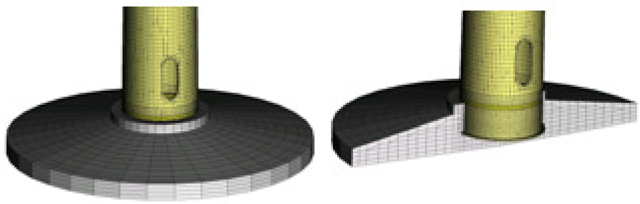

Figure 2. Foundation of the tower (FE model)

The reinforced concrete foundation is modeled together with the tower body. The whole system is assumed to be elastically supported to the foundation base taking account the soil-structure interaction. The foundation has been introduced by means of brick elements, elastically supported to the ground, through unilateral contact and friction conditions.

2.2 *Loading*

The self-weight of the tower itself is 1422 kN. The weight of the nacelle, including the blades and the rotor as provided by the manufacturer is equal to G_r=1067 kN, having the center of gravity shifted horizontally +0,725 m from the axis of the tower and vertically +0.50 above the upper flange level. The wind loads of the tower are divided into two parts; the loads at the top of the tower and the loads over the tower stem. The loads on the tower top are provided by the manufacturer for various accidental cases. In the present work, one of the most unfavorable pair of loads will be used (V_{top}=598.74 kN, M_{top}=1665,41 kNm). The loads over the tower stem are calculated (EN 1993-1-4) for a basic wind velocity at 10m above ground of v_b=27,0 m/sec and for a terrain of category II (Figure 3).

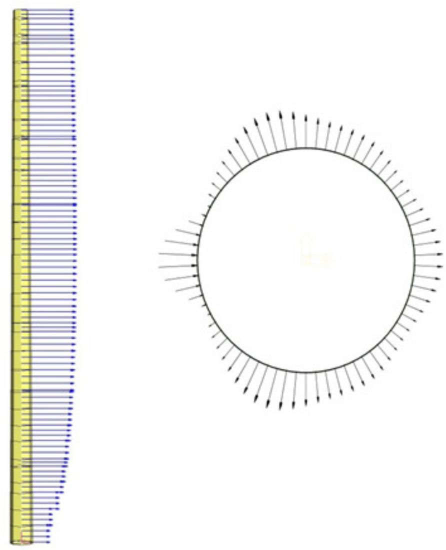

Figure 3. Stem loading over the height and the circumference of the tower

3 ANALYSIS OF THE TOWER

3.1 *Simplified model*

The tower being a cantilever beam (Figure 4) is a statically determinate system, so shear force and bending moment at every point could be easily calculated by hand. In contrast, calculation of the tower deformations is quite difficult to be performed by hand due to the varying moment of inertia along the height of the tower.

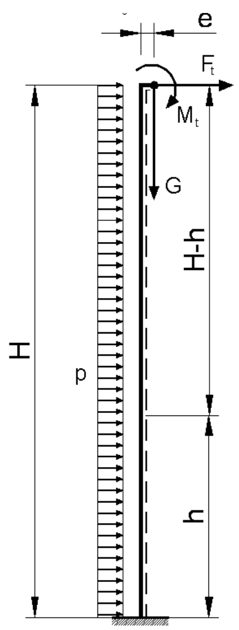

Figure 4. Simplified structural model

For the self-weight, the reactions V,M at the base are:
V=0,0 kN , M=1.067•0,725=773,60 kNm (due to the eccentricity of the load)
The total weight of the tower (including flanges & stiffeners) is: 1.422 kN
Wind loads on the tower top are producing the following reactions:
V=598,74kN ,
M=1.665,41+598,74*(76,15+ez)=47.558,83 kNm
The corresponding wind forces over the height are (EN 1993-1-4):
Fw1(z) = 0,51•(-0,01775•z+4,30266) , z < 2,0 m
Fw2(z) = 0,013n(20•z)•[ln(20•z) + 7]•(-0,01775•z + 4,30266) , z >= 2,0 m

Base reactions are resulting from the integration of the load functions:

$$V = \int_{z=0}^{2.0} Fw1(z) \cdot dz + \int_{z=2}^{76.15} Fw2(z) \cdot dz$$

and

$$M = \int_{z=0}^{2.0} Fw1(z) \cdot z \cdot dz + \int_{z=2}^{76.15} Fw2(z) \cdot z \cdot dz$$

The integrations above give as a result: V=302,56 kN & M=12.027,36 kNm.
The total wind forces at the tower base are:
N=1422,00+1.067,00=2.489,00 kN
V=598,74+302,56=901,30 kN
M=47.558,83+12027.36=59.587 kNm
Similarly, for the load combination: [G+1.50·W] :
N=2489,00 kN
V=0,00+1,5• 901,3=1.351.95 kN
M=1,5•59.586,19-773,58=88.606 kNm

3.2 *Analysis for extreme wind loading*

The overall FE model for the extreme wind loading has been analyzed using GMNIA (Geometric & Material Non-linear with Imperfections Analysis). To the linear model, an LSA (Linear Static Analysis) has been performed. The section forces for the load combination G+1,50W at the tower base are: N=2.453 kN, V=1.342 kN, M=88.731 kNm (as expected, almost identical to both models and hand calculation). The displacement at the top of the tower for the load combination G+W is 1,195 m. For the same combination, an uplift of about 20% of the foundation is recorded (Figure 5). As expected, since the tower is a statically determinate system, the cross-section forces are almost identical for both calculation models and for hand calculation as well. The main difference in the results, between the linear and the FE model is caused by the wind distribution along the tower circumference which cannot be described by the linear model. Note that in the FE model, the distribution of the wind forces over the circumference are causing on-plane deformation of the tower section. The fact that the section is forced to remain circular on the flange positions due to the stiffness of the flanges, causes concentration of circumferential stresses in the vicinity of the flange positions (Figure 6).

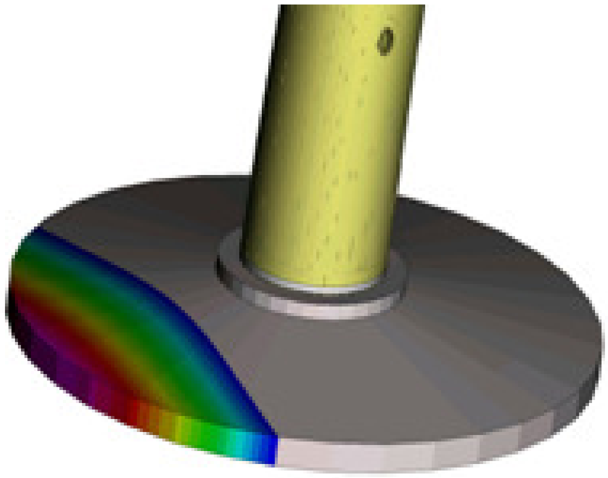

Figure 5. On the uplift of the foundation

Compressive circumferential stresses also appear to the vicinity of the door due to the existence of the stiffening ring around the door opening. The above compressive stresses that cannot be described by the use of a linear model or by hand calculation, are participating to the local buckling of the shell on the above positions (Lavassas et al. 2003).

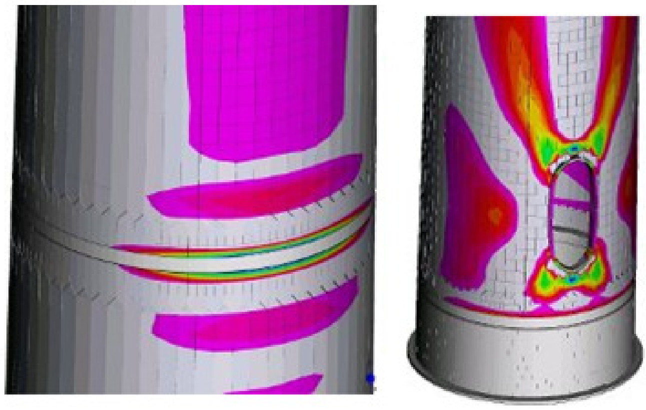

Figure 6. Compressive circumferential stresses at the vicinity of the flanges and at the door opening (G+1,50W)

In addition to the GMNIA analysis, a linear buckling analysis has been also performed. Global and local imperfections to the model for the GMNIA and LBA analyses are introduced according to the specifications of EN 1993-1-6. The first buckling eigenvalue for imperfection class B has been found equal to 3.44 corresponding to the local shell elastic buckling at about 6/8 of the tower height. The first 10 positive eigenvalues are close enough to the first one being located to the upper part of the tower (Figure 7). Negative eigenvalues are obviously neglected.

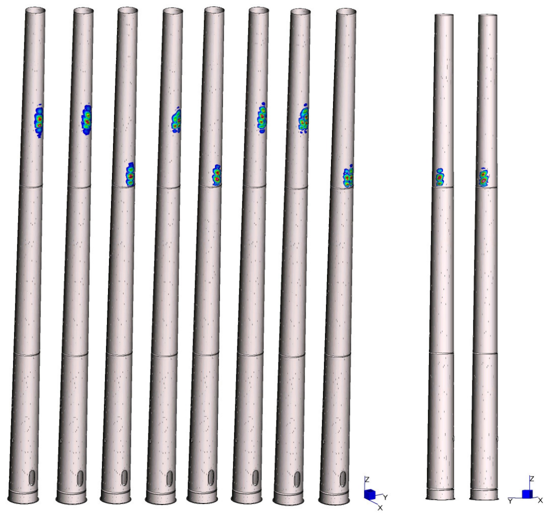

Figure 7.Eigenmodes [1] ÷ [5] , [7] , [8] , [10] (positive) & [6] , [9] (negative)

3.3 *Seismic Analysis*

The wind action is in most cases the dominant loading for the design of a wind turbine tower. However, the turbine-tower system as a flexible structure having a big height and the mass of the rotor system and the blades concentrated at the top forms an inverse pendulum. Designing the tower for an area with high seismic risk, it must be analyzed for the seismic loads as well, in order to determine whether the latter is significant for the design of the structure or not.

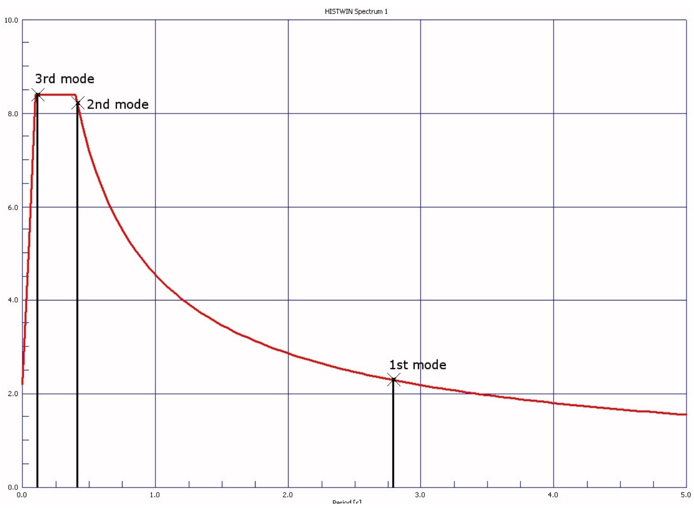

Figure 8. Design spectrum applied to the tower (the three governing eigenvalues are noted)

The seismic actions have been determined on the basis of the linear-elastic behavior of the structure with bilateral support to the ground, whereas the reference method is the modal response spectrum analysis (EN 1998-1) & (EN 1998-6). Firstly, an eigenvalue analysis was performed for both models. The total participating mass without considering the contribution of the foundation is about 93% in both directions. The governing eigenvalues and eigenshapes are shown on Table 1 and Figure 9.

Table 1. Eigenvalues of the tower.

Eigenfrequency (Hz)	FE model	Linear model	Mass participation
1st	0,357	0,324	61,7%
3rd	2,820	2,626	14,9%
9th	7,520	7,850	4,9%

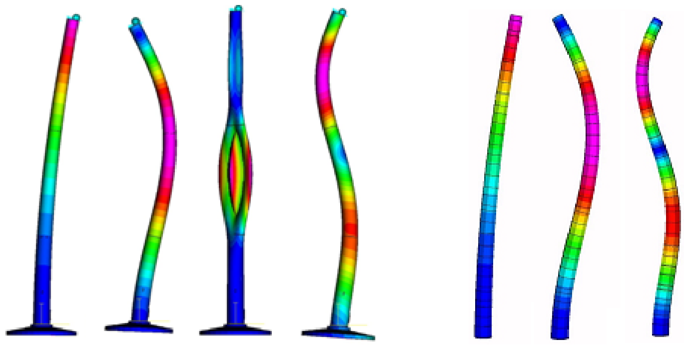

Figure 9. Eigenshapes: FE model (left), Linear model (right)

The relevant earthquake data for performing the spectrum analysis are as follows:

$a=0.24$, $T_B=0.10$, $T_C=0.40$, $\gamma_I=1.40$, $\zeta=2\%$

For the assessment of the effects due to the combination of the components of the seismic action, the SRSS procedure was adopted. A comparison of the seismic loading results versus the relevant for extreme wind loading is shown in Table 2.

Table 2. Displacements & forces for wind and seismic loading

Description	G+W	G+1,5W	G+E
F,top (mm)	1195		530
V,Base (kN)		1.342	499
M,Base (kN)		88.731	25.449

As shown, the earthquake internal forces are less than 30% of those corresponding to the extreme wind. For this type of seismic data, the critical loading for the design of the tower is the extreme wind. Even after combining the seismic load with the operating wind (GL Wind 2003) it gives a stress state of about 75% of the corresponding for the extreme wind for the specific seismic data.

4 LIMIT STATE DESIGN

Having on hand the analysis results, the tower is designed for plastic (LS1) and buckling (LS3) limit states. For the plastic limit state, the maximum von Mises stress at any point of the tower is compared to the yield limit of the steel. According to the Finite

Element model results, the maximum von Mises stress on the tower shell has been found equal to 348 MPa, in the vicinity of the door. For the main body of the tower, the maximum von Mises stress is lower, reaching 293 MPa. It is worth-mentioning that there is an almost uniform distribution along the 2/3 of the tower height. Regarding the Linear Model, the corresponding maximum stress is 251 MPa. In this case, special models to the tower details (flange connections, door position) are needed for examining the stress state on those positions.

For the buckling design of the tower shell, three methods are proposed (EN 1993-1-6):

1. Design by global numerical analysis using GMNIA analysis (§8.7)
2. Design by global numerical analysis using MNA and LBA analyses (§8.6).
3. Stress design (§8.5).

The first two methods require advanced calculations and in particular, a Limit Load Analysis with or without imperfections and a Buckling analysis. The third one requires linear elastic calculations.
The limit load of the tower is calculated for the perfect shell and for two of the three types of local imperfections proposed by EN 1993-1-6.

Out-of-roundness imperfection has been also applied to the whole shell. In this case, each cross section of the shell was forced to form an elliptical shape having the biggest diameter vertical to the direction of the wind force. For the application of this type of imperfection, a constraint has been applied to every node of the tower, following a function that causes the elliptical deformation. The deformed shape of the application of the constraint to the tower forms the imperfect tower shell (Figure 10), i.e.

$$d_{max}-d_{min} = 0{,}010 \cdot d$$

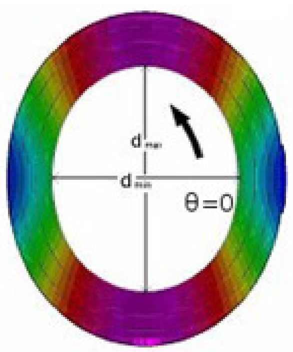

Figure 10. Out-of-roundness imperfection of the tower (top view - magnified)

Regarding the dimple imperfection, since it can occur anywhere on the tower body, it is difficult to predict the exact location of the imperfection in order to be the most unfavorable for the analysis of the tower, and it is practically impossible to fill the whole tower with dimples. It was decided a dimple imperfection to be put to the position of the 1st buckling eigenmode (Figure 11) This type of imperfection is applied to the tower by magnifying the amplitude of the eigenmode until the deformation is the maximum allowed (EN 1993-1-6) for fabrication class B ($U_{0,max} = 0{,}010$).

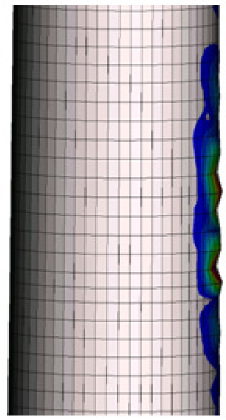

Figure 11. Dimples generated from the 1st eigenmode of the perfect shell (magnified)

Accidental eccentricity imperfection has also applied to the position of the 1st buckling eigenmode for the same reasons as above. Accidental eccentricity is introduced to the model by the application of a rigid offset to the specific elements i.e. $e_a = 3$mm.

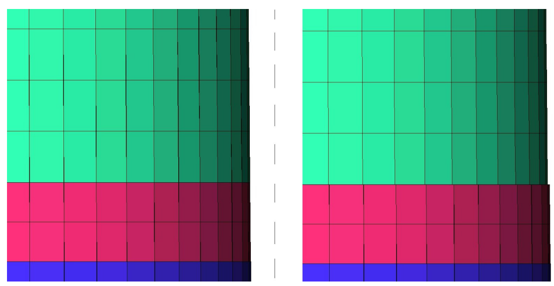

Figure12. Perfect shell (left), shell with accidental eccentricity (right)

The limit load factor is determined by an iterative Geometrically and Materially Nonlinear Analysis by means of step-by-step increments of the wind load until the failure of the structure. The limit load analysis resulted to a limit load factor: $r_{Rk} = 2{,}05$ for the perfect shell and $r_{Rk} = 1{,}95$ for the imperfect. Figure 13 shows the rotation at a point near to the position of the door during the load steps until the collapse of the structure occurs. The tower reaches the limit load due to shell buckling at the plasticized areas near the door position (Figure 14). The corresponding Buckling Resistance ratios are 1,42 for Case 1, and 1,32 for Case 2.

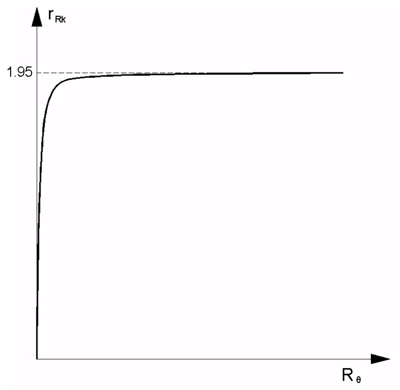

Figure 13. Shell buckling at the limit load of the tower

Shell buckling occurs also near to the position of the flanges due to the concentration of circumferential stresses.

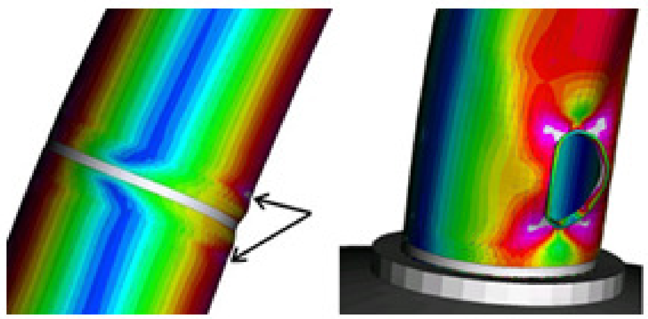

Figure 14. Shell buckling at the limit load of the tower

Case 3 cannot be applied to an unstiffened shell. Taking into account as stiffened the parts between two connection flanges, the allowable stresses in the circumferential direction are very small comparing to the ones obtained near the flange positions.

5 CONCLUSIONS

A linear model can be used for the seismic design of the tower, but not for the design for wind actions because of the presence of circumferential stresses near the flange positions.

The dominant load for the design of the specific tower is the extreme wind loading. However, it is not evident that this conclusion stands for all cases of wind tower design. An analysis for seismic loads must be done in all cases in order to determine whether it is significant or not for the structural design.

The results from the limit load analyses indicate that the tower is reaching the plastic limit load before approaching the elastic bifurcation point. The collapse is induced by the shell buckling at the material yielding zones, where the plasticizing von Mises stresses are components of the compressive meridional and circumferential stresses.

The preferable method for the design of the tower against buckling seems to be the global numerical analysis using LBA and MNA analyses, according to EN 1993-1-6 §8.6. In this case the imperfections are introduced indirectly by the employment of the overall elastic imperfection factor (r_{Rov}).

The global numerical analysis using GMNIA analysis design according to EN 1993-1-6 §8.7 is more straightforward, but at the same time it is proved to be more tedious and requires an in-depth knowledge of the applicable imperfections and the calibration factor (k_{GMNIA}).

The stress design procedure of EN 1993-1-6 §8.5 results in rather conservative values, especially considering the circumferential stresses. The use of stiffening rings for this type of analysis is inevitable (Lavassas et al. 2003).

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