

RFS-CT-2006-00031 - HISTWIN
High-Strength Steel Tower for Wind Turbine

**WP3.2 – EFFECTS OF GEOMETRICAL IMPERFECTIONS
TO THE RESISTANCE OF THE TOWER**

BACKGROUND DOCUMENT

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1. WORK PACKAGE DESCRIPTION

WP leader: AUTH

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Task: Numerical analysis of effects of geometrical imperfections on the resistance of the tower.

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2. BACKGROUND DOCUMENT

2.1. General aspects

1. The effects of geometrical imperfections on the tower are being investigated, as proposed by EC-3-1-6 by means of linear buckling analysis (LBA) and geometrical & material non-linear analysis with imperfections (GMNIA). Finally design of the tower by the three methods proposed by EC3-1-6 is attempted.
2. As proposed by EC3-1-6 there are three fabrication classes in the production:
 - Class A : Excellent
 - Class B: High
 - Class C: Normal

The tower is constructed by REPOWER using fabrication class B. However the influence of all fabrication class tolerances will be investigated.

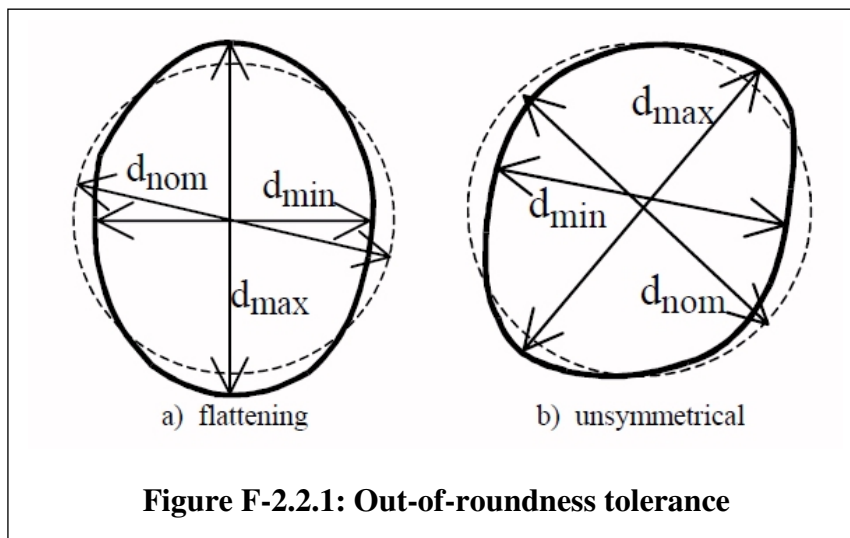
2.2. Geometrical imperfections – [EC 3-1-6] specifications

The main types of imperfections proposed by EC3-1-6 are classified as follows:

2.2.1. Out-of-roundness tolerance

1. The out-of-roundness tolerance is determined in terms of the parameter U_r :

$$U_r = \frac{d_{\max} - d_{\min}}{d_{\text{nom}}}$$



Where:

$d_{\max}$ and $d_{\min}$ are the maximum and minimum measured internal diameters

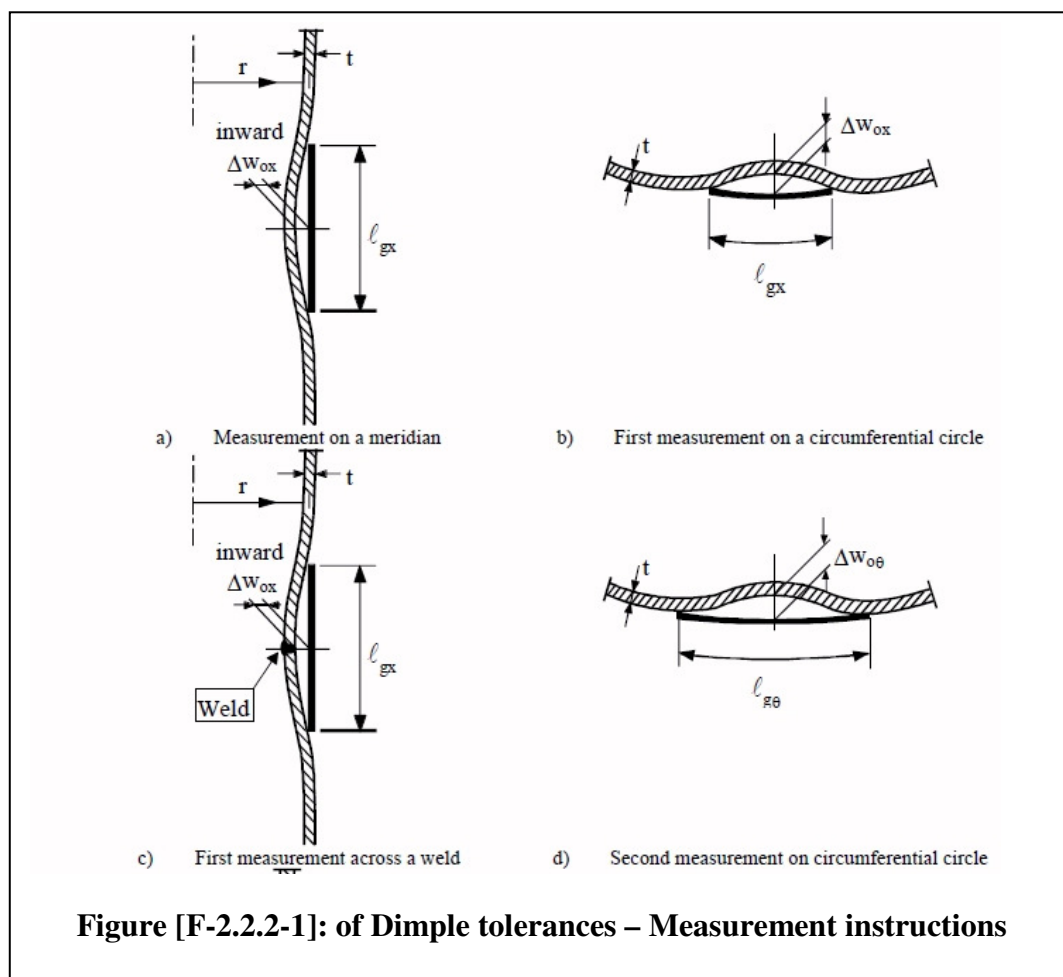
d_{nom} is the nominal internal diameter

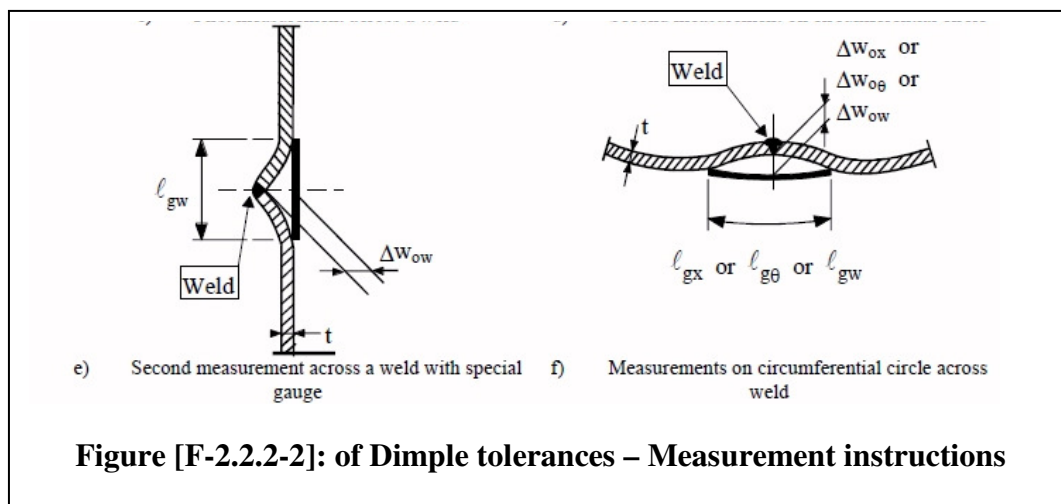
2. The recommended values for the of-roundness tolerances are given in Table [T-2.2.1], as derived by [EN 3-1-6 Table 8.1]. Our tower is assumed to be of fabrication class B, having a diameter larger than 1,25 m in all the sections, so the recommended value for the out-of-roundness tolerance is 0,010 m.

Table [T-2.2.1]: Of-roundness tolerance parameter $U_{r,max}$				
Fabrication tolerance class	Diameter range	$d \leq 0,50m$	$0,50m \leq d \leq 1,25m$	$1,25m \leq d$
	Description	Value of $U_{r,max}$		
Class A	Excellent	0,014	$0,007+0,0093 \bullet (1,25-d)$	0,007
Class B	High	0,020	$0,007+0,0133 \bullet (1,25-d)$	0,010
Class C	Normal	0,030	$0,007+0,0300 \bullet (1,25-d)$	0,015

2.2.2. Dimple tolerances

A dimple measurement gauge should be used in every position for both the meridional





and circumferential directions (see Figure [F-2.2.2]). The dimple tolerance parameter is given by the formula:

Table [T-2.2.2]: Dimple tolerance parameter $U_{r,max}$		
Fabrication tolerance quality class	Description	Value of $U_{0,max}$
Class A	Excellent	0,006
Class B	High	0,010
Class C	Normal	0,016

$$U_{0,max} = \frac{\Delta_{w0}}{l_g}$$

and the recommended values for the relevant fabrication classes are given in Table [T-2.2.2], as derived by [EN 3-1-6 Table 8.4]:

As mentioned above, for the specific tower it is assumed Class B, so the recommended value for $U_{0,max}$ is equal to 0,01 m.

2.2.3. Accidental eccentricity imperfections

Those are imperfections in which two jointed parts are placed accidentally so their theoretical mid-plane axes are not continuous as in the theoretical model, but having a specific eccentricity (see Figure [F-2.2.3]). The values for the maximum allowable eccentricity for each fabrication class are given in Table [T-2.2.3], as derived by [EN 3-1-6 Table 8.2]:

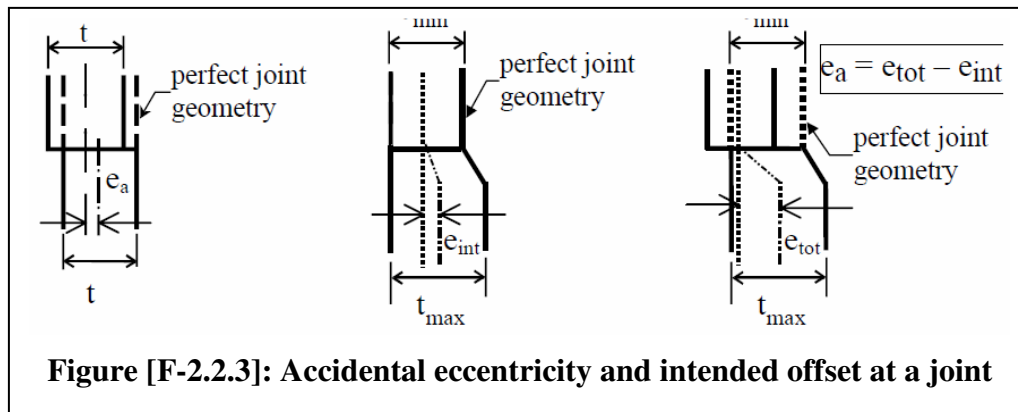


Table [T-2.2.3]: Accidental eccentricities $e_{a,max}$		
Fabrication tolerance quality class	Description	Maximum permitted accidental eccentricity
Class A	Excellent	$e_a \leq 2\text{mm}$
Class B	High	$e_a \leq 3\text{mm}$
Class C	Normal	$e_a \leq 4\text{mm}$

Similarly, the recommended value for our tower is $e_a \leq 3\text{mm}$

2.3. Impact of the imperfections to the resistance of the tower

The investigation of the effects of the various types of imperfections to the resistance of the tower requires the introduction of the imperfect geometry to the calculation model. For this scope the steps below have been followed :

- Built an analytical Finite Element model for the whole structure (perfect geometry)
- Perform buckling analysis for the perfect geometry model
- Introduce the fabrication tolerances to the Finite Element model, leading thus to an imperfect geometry model
- Perform buckling analysis for the imperfect geometry model

- Compare the analysis results

2.3.1. Buckling analysis of the perfect shell

A linear buckling analysis (LBA) has been performed to the perfect shell for the extreme wind load combination [1,0·G+1,50·W]. The tower geometry and the materials, conform with the REPOWER Systems drawings: [R050405- -SZ (B)] & [R050405-EZ (D)]. The tower height is 76,15m. Its diameter varies from 4,300 m at the base to 2,955 m to the tower top. Shell thicknesses are varying from 30 mm at the base to 12 mm near to the tower top. The extreme wind loading for which the buckling analysis is performed, is described on WP 3.1 §2.3 .

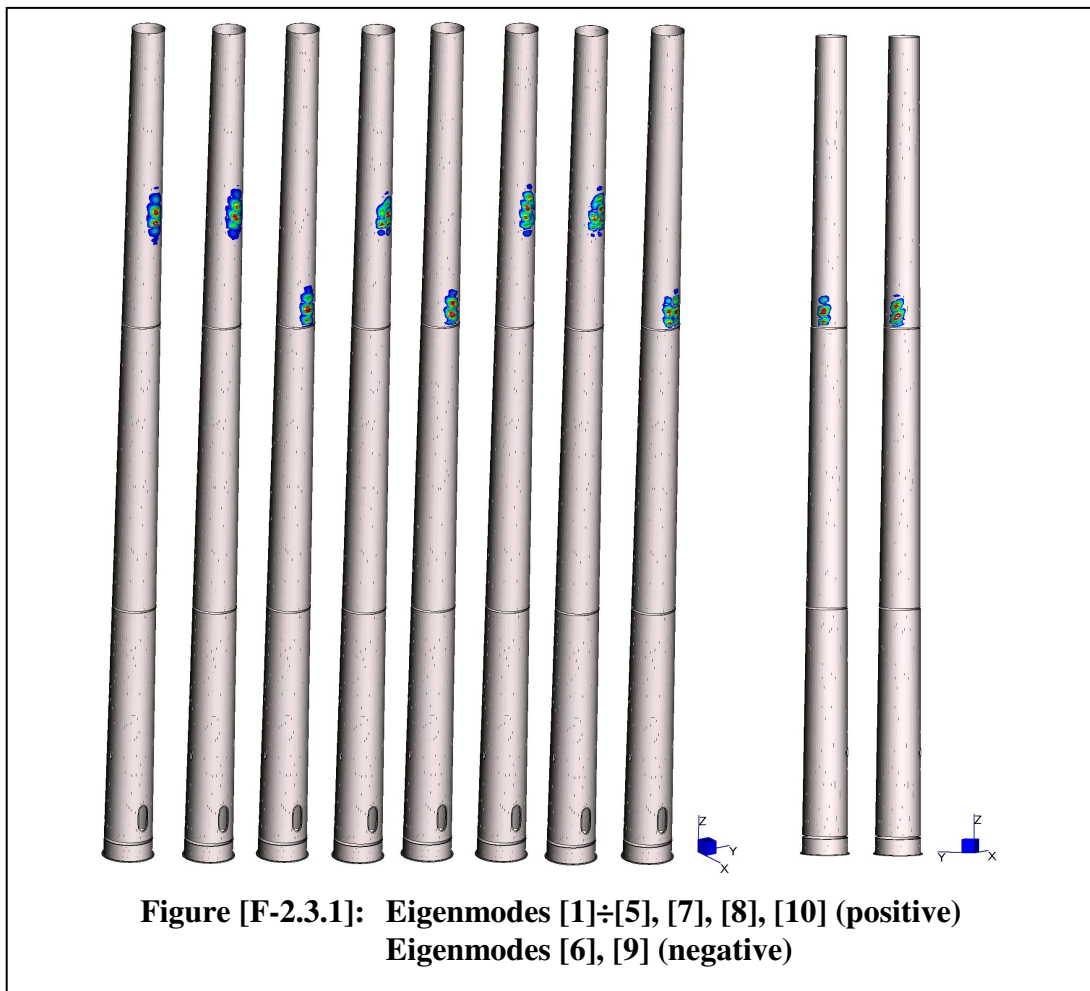
The first 10 buckling eigenvalues are calculated, as presented in Table [T-2.3.1] and Figure [F-2.3.1]. The linear buckling eigenvalues represent the factors [r_{Rcr}] determined by the expression:

$$r_{Rcr} = F_{Rk} / F_{Ed}$$

where [F_{Ed}] represent the design loads and [F_{Ed}] the characteristic buckling resistance, at the bifurcation point.

1. All buckling eigenvalues are located to the upper part of the tower, specifically on courses [27] & [22]. Eigenvalues [6] & [9] are negative, and therefore they correspond to the reverse load direction.

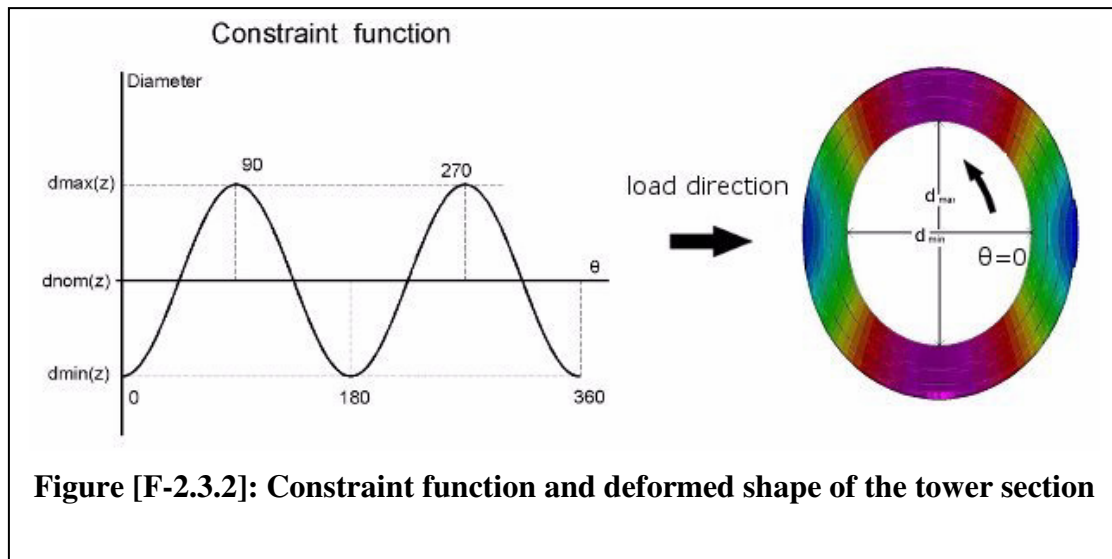
Table [T-2.3.1]: Buckling eigenvalues [r_{Rcr}]										
Eigenmode	[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]	[10]
Eigenvalue	3.44	3.53	3.57	3.60	3.62	-3.65	3.66	3.69	-3.72	3.73



2.3.2. Out of-roundness imperfections

1. The introduction of the out-of-roundness imperfections to the Finite Element model is achieved by the enforcement of the nodes of the perfect geometry structure to the desired positions, by means of appropriately defined constraints. At a next stage, the nodal displacements are added to the initial node coordinates, producing thus the imperfect shape of the shell.
2. For the specific case, the circular cross-sections of the tower have to be ovalized, that is they must be transformed to egg-type shapes, having the maximum diameter perpendicular to the load direction, so that the maximum radius of curvature on the expectant buckling position of the shell is attained.
3. The application of a sinusoidal variated constraint to all nodes of the shell has been adopted as the most suitable method to accomplish the above deformation of the cross-sections. This constraint function forces the nodes to reach the minimum axis

of the section $[d_{\min}]$ at positions $\theta=0^\circ$ & $\theta=180^\circ$ and the maximum axis $[d_{\max}]$ at $\theta=90^\circ$ & $\theta=270^\circ$ (see Figure [F-2.3.2]).



4. Four computational models have been built:

- Perfect shell
- Imperfect shell – Class A fabrication tolerance: $d_{\max}-d_{\min} = 0,007 \cdot d_{\text{nom}}$
- Imperfect shell – Class B fabrication tolerance: $d_{\max}-d_{\min} = 0,010 \cdot d_{\text{nom}}$
- Imperfect shell – Class C fabrication tolerance: $d_{\max}-d_{\min} = 0,015 \cdot d_{\text{nom}}$

For example, at the tower base ($z=0$, $d_{\text{nom}}=4,3$ m) for fabrication class B we have:

$$d_{\max}-d_{\min} = 0,010 \cdot 4,3 = 0,043 \text{ m}$$

5. The application of the above function to the tower is as follows: At a specific height, every point should have a diameter:

Table [T-2.3.2]: Out-of-Roundness imperfections – Buckling eigenvalues $[r_{\text{Rcr}}]$				
Eigenmode	Perfect shell	Class A imperfection	Class B imperfection	Class C imperfection
[1]	3.44	3.43	3.40	3.39
[2]	3.53	3.51	3.49	3.49
[3]	3.57	3.55	3.53	3.54
[4]	3.60	3.59	3.56	3.56
[5]	3.62	3.61	3.58	3.57

$$D(z,\theta)=d_{\text{nom}}(z) + a \cdot \sin (\theta-\pi/2) \quad (1)$$

where $a = (U_{r,\text{max}} \cdot d_{\text{nom}}(z)) / 2$

having so at $\theta=0$ & $\theta=\pi$:

$$d(z,[0|\pi])=d_{\text{nom}}(z) - a$$

and at $\theta=\pi/2$ & $\theta=3 \cdot \pi/2$

$$d(z,[\pi/2|3\pi/2])=d_{\text{nom}}(z) + a$$

The tower diameter varies lineary along its height and can be expressed as a function of z (WP3.1 §2.3.2):

$$d_{\text{nom}}(z) = -0,01775 \cdot z + 4,30266$$

Substituting the values of a & $d_{\text{nom}}(z)$ equation (1) takes the form:

$$D(z,\theta)= -0,01775 \cdot z + 4,30266 + (U_{r,\text{max}} \cdot (-0,01775 \cdot z + 4,30266)) / 2 \cdot \sin (\theta-\pi/2)$$

Each node of the tower is being forced by a constraint

$$dr(z,\theta)=(D(z,\theta)- d_{\text{nom}}(z)) / 2$$

The equation above is being applied by the FE software Strand7 to all the nodes of the structure in cylindrical coordinates. After applying the constraints, the deformed shape of the tower is being used as the new tower geometry with the out-of-roundness imperfections introduced.

6. A linear buckling analysis is performed next for the four models and the first 5 eigenvalues are compared to the one corresponding of the perfect shell. As it is demonstrated in Table [T- 2.3.2], the buckling eigenvectors of the perfect and imperfect shell configurations are almost identical, the deviations being insignificant. Although usually the first eigenvalue is the most significant, in the case of the wind turbine tower the structure is a long cylinder and the various eigenmodes correspond to the local buckling of the shell on different courses of the tower. If the first eigenvalue has a significant difference from the second, the third, etc, it is an indication that there is a weak course on the tower (where the 1st eigenmode occurs). For the above reason the first 5 eigenvalues are presented.

2.3.3. Eccentricity imperfections

1. Accidental eccentricity imperfections can be introduced to the Finite Element model by the application of a rigid offset to the elements. It is obvious that the application of this deformation to the entire tower shell is meaningless, since this will practically eliminate the imperfections. Therefore, a specific course (or a group of non-adjacent courses) must be selected for the application of the rigid offset to the elements. In our case, courses [27] & [22], on which the first buckling eigenvalues appear for the perfect shell, are selected (see Figure [F-2.3.3-1]).

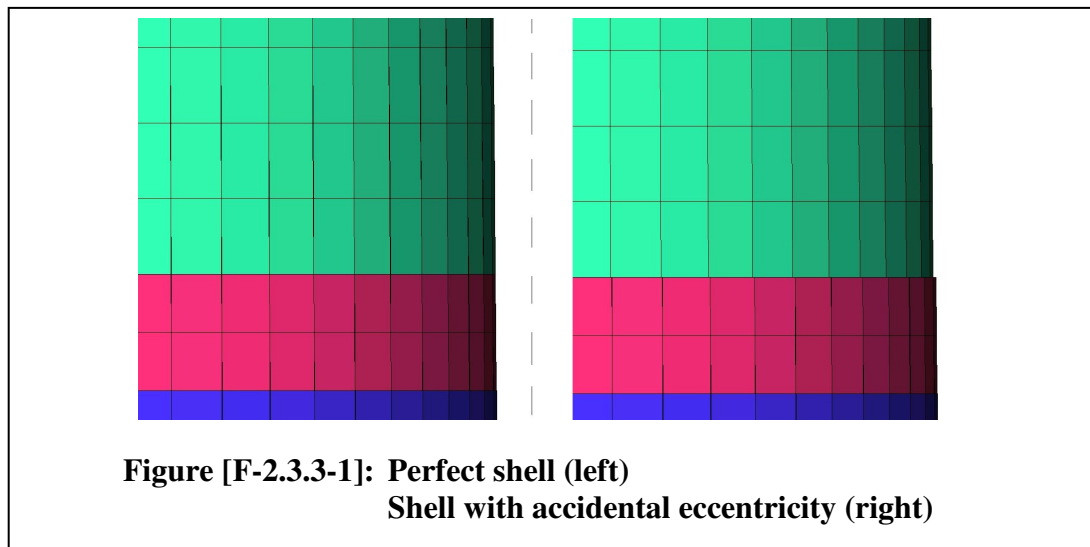
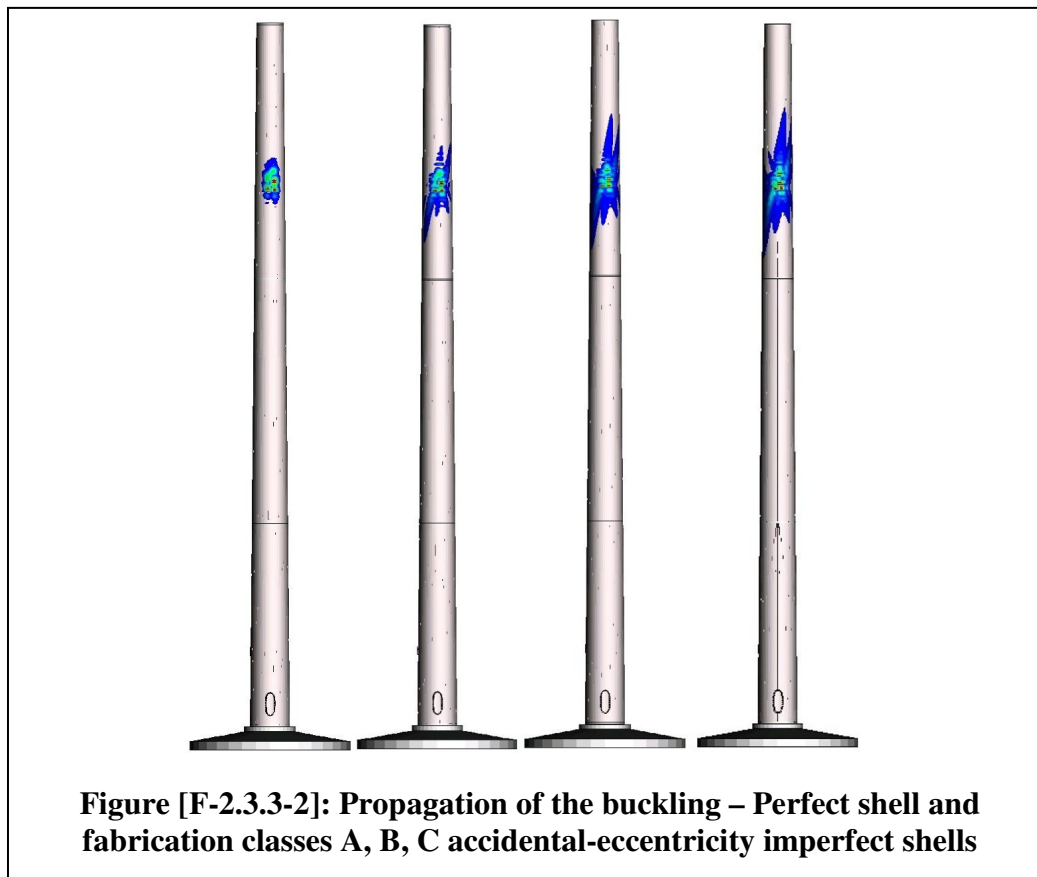
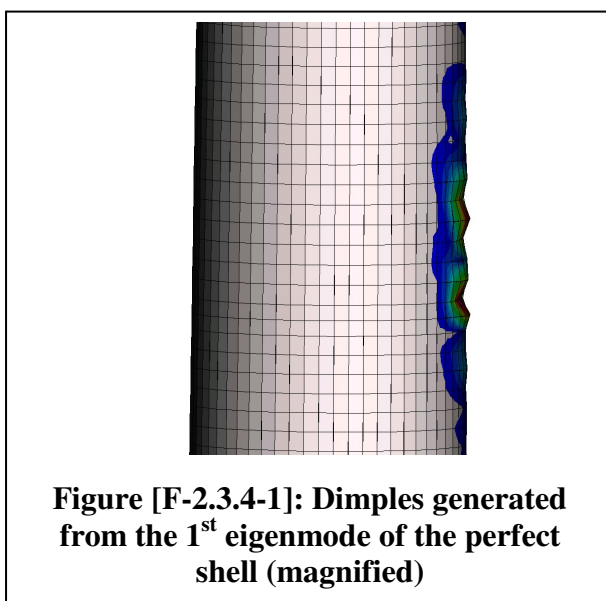


Table [T-2.3.3]: Accidental eccentricity imperfections – Buckling eigenvalues [r_{Rcr}]				
Eigenmode	Perfect shell	Class A imperfection	Class B imperfection	Class C imperfection
[1]	3.44	3.43	3.42	3.40
[2]	3.53	3.52	3.51	3.49
[3]	3.57	3.56	3.56	3.54
[4]	3.60	3.58	3.56	3.55
[5]	3.62	3.62	3.62	3.61

2. As demonstrated in Table [T-2.3.3], there is no notable difference between the 5 first eigenvalues. By the comparison of the eigenmodes though, it is evident that, in contradiction to the restriction of the perfect model buckling within the specific course, the corresponding imperfect models present a propagation of the buckling to the neighboring courses (see Figure [2.3.3-2]).



2.3.4. Dimple imperfections



1. The introduction of the dimple imperfections to the Finite Element model is a rather laborious task, due to the fact that, having by nature a random distribution, they can appear anywhere along the tower body.
2. Hence, an approach analogous to the one described in [§2.3.3-1] is performed and specifically, a dimple imperfection in the position where the 1st eigenmode occurs in the perfect shell will be generated.
3. For this scope, the following procedure are carried out:

- Execute a buckling analysis on the perfect shell for the extreme wind combination
- Examine the 1st buckling eigenmode
- Measure the length of main dimple on the 1st buckling eigenmode
- Multiply the 1st buckling eigenmode results by a factor, in order to accomplish the dimple tolerance parameter suitable for each fabrication class
- Add multiplied nodal displacements to the node coordinates
- Perform a buckling analysis to the imperfect structure in order to find the buckling eigenvalue of it, or perform a GMNIA calculation for the extreme wind load combination.

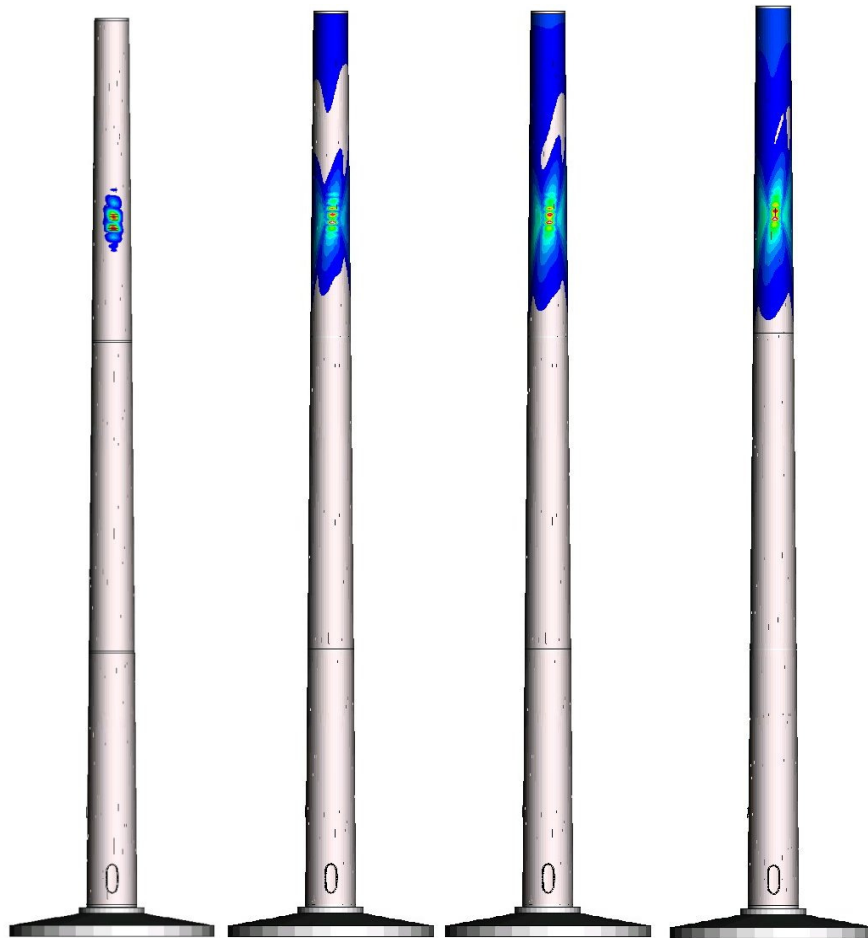


Figure [F-2.3.4-2]: 1st buckling eigenmode – Perfect shell and fabrication classes A, B, C dimple imperfect shells (perfect shell & fabrication classes A,B,C)

Table [T-2.3.4]: Dimple imperfections – Buckling eigenvalues [r_{Rcr}]				
Eigenmode	Perfect shell	Class A imperfection	Class B imperfection	Class C imperfection
[1]	3.44	3.42	3.40	3.36
[2]	3.53	3.53	3.52	3.47
[3]	3.57	3.57	3.57	3.57
[4]	3.60	3.59	3.57	3.57
[5]	3.62	3.62	3.62	3.62

4. The comparison of results is given in Table [T-2.3.4]. It must be noted that eigenvalues [3] & [5] are not affected, because they correspond to course [S22] buckling.

2.4. Assessment of the limit load for the tower

1. The bifurcation load factor for the perfect shell is derived from the Linear Buckling Analysis (LBA) and equals to: $r_{Rcr} = 3.44$. Similarly, 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. It is noted that for the model under consideration, the unilateral contact conditions are removed and the contact of the foundation base to the ground is considered as bilateral elastic. This is an inevitable intervention, in order to prevent the overturning of the tower as a whole, before it reaches the buckling or plastic limit state (even for the design combination: $[G+1.50 \cdot W]$ there is a 40% uplift of the foundation).
2. Having in mind that the specific approach requires a noteworthy computational effort, only three of the various cases are investigated:
 - Case [1] : Shell with Class B out-of-roundness tolerances, introduced globally to the tower
 - Case [2] : Shell with Class B dimple tolerances, introduced to the location of the 1st eigenmode
 - Case [3] : Perfect shell

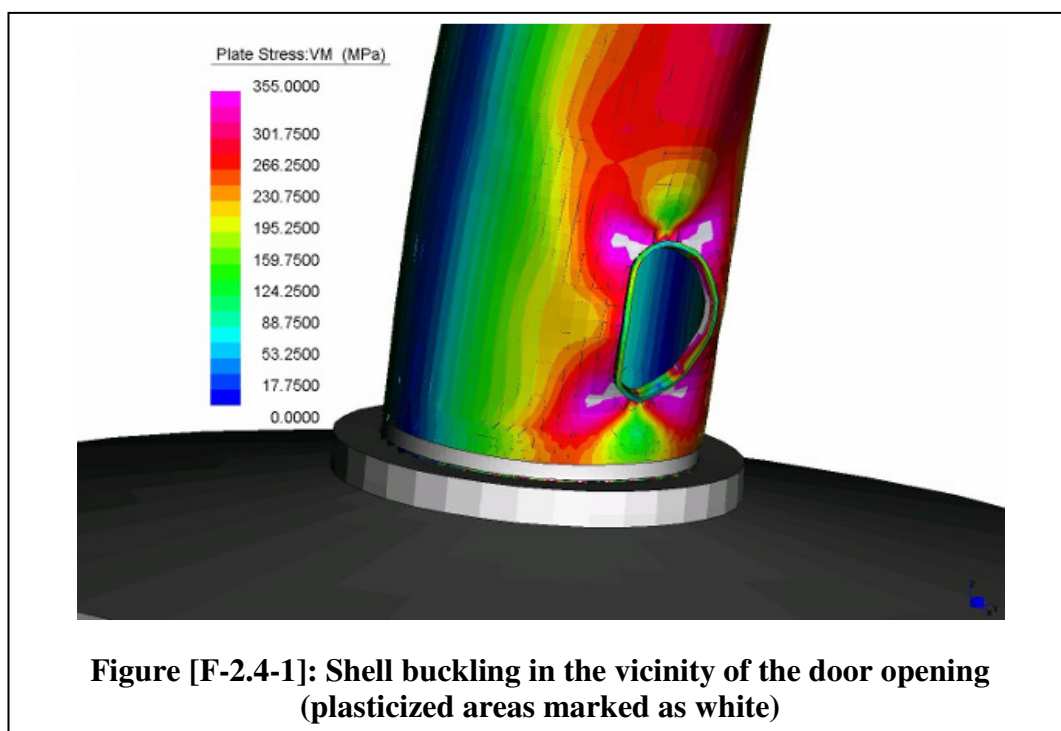


Figure [F-2.4-1]: Shell buckling in the vicinity of the door opening (plasticized areas marked as white)

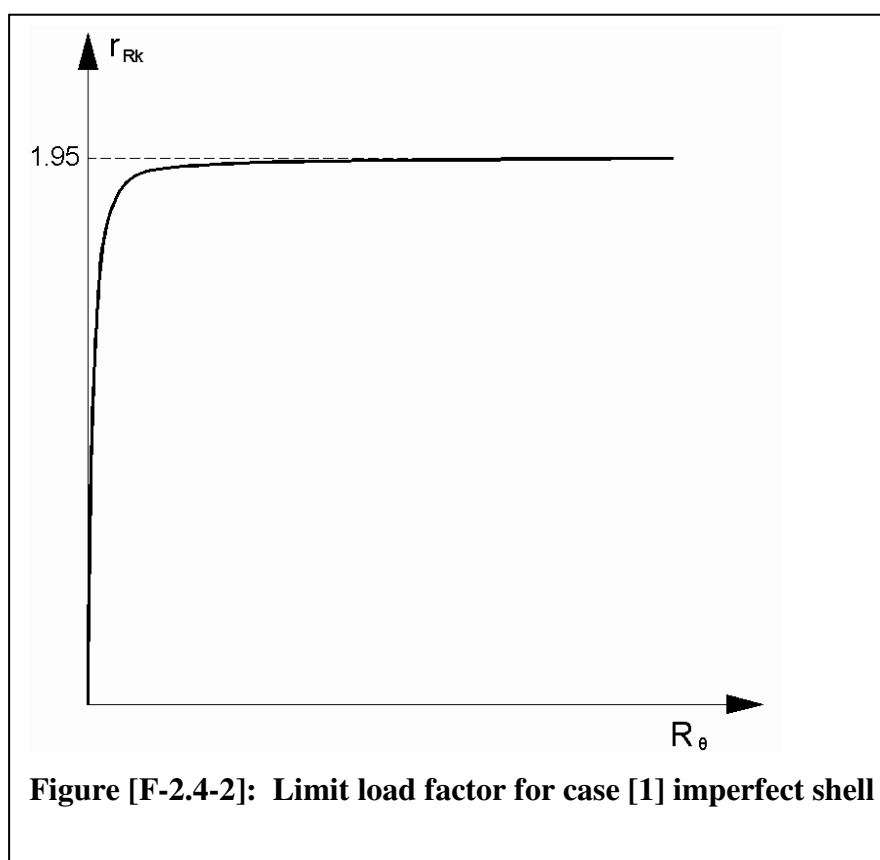
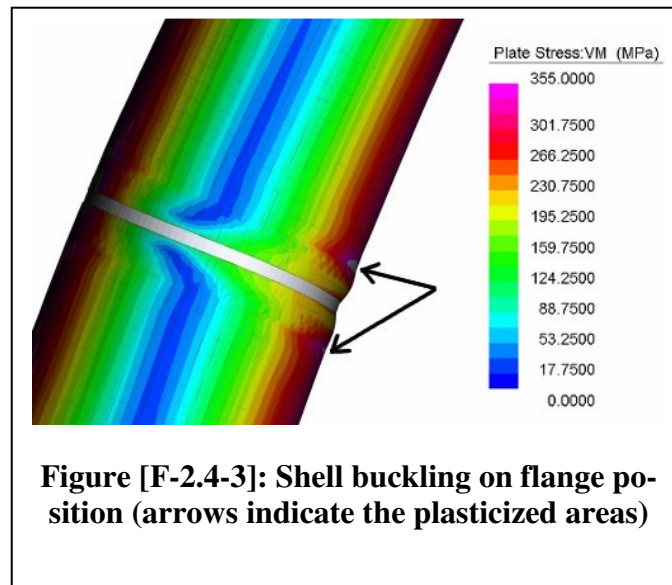


Figure [F-2.4-2]: Limit load factor for case [1] imperfect shell



3. Case [1]: The procedure diverged for a characteristic resistance ratio (limit load factor): $r_{Rk} = 1,95$, which corresponds to the load combination: $[1.0 \cdot G + 1,95 \cdot W]$. Fig. [F-2.4-2] shows the evolution of the rotation R_0 of a point right above the door position. The failure of the structure was due to the buckling at the location of the door, induced by the excessive incremental displacements. As shown in Figure [F-2.4-3], the buckling of the shell takes place at the plasticized areas, when the modulus of Elasticity was significantly reduced. In other words, the shell has entered into the plastic state, before reaching the bifurcation point. In addition, shell buckling occurs at the vicinity of the flanges, due to the presence of high circumferential stresses, in conjunction with the meridional ones.
4. Case [2]: Despite the fact that the dimple imperfection has been introduced to the location of the 1st buckling eigenmode, the shell eventually has buckled around the door opening, for the same reason as in Case [1]. The characteristic resistance ratio was found in this case equal to: $r_{Rk} = 2,05$.
5. Case [3]: The analysis for the perfect shell results in an identical limit load factor: $r_{Rk} = 2,05$.

2.5. Buckling analysis according to [EC 3-1-6]

Code [EC 3-1-6] refers to three distinguished calculating procedures for the buckling assessment of the tower:

2.5.1. [EC3-1-6 §8.7]: Design by global numerical analysis using GMNIA analysis

1. This is without doubt the most advanced design procedure suggested by the Eurocode. The imperfect elastic-plastic buckling resistance ratio $[r_{R,GMNIA}]$ represents the load factor, derived as the lowest value obtained by the use of the criteria C1, C2 and C3, as follows:
 - Criterion C1: The maximum load factor on the load-deformation load (limit load)
 - Criterion C2: The bifurcation load factor, where this occurs during the loading path before reaching the limit point of the load-deformation curve
 - Criterion C3: The largest tolerable deformation, where this occurs during the loading path before reaching a bifurcation load or a limit load. But since no data have been supplied by the manufacturer for the investigation of [C3] and therefore this check will be omitted at this stage.
2. The characteristic buckling resistance ratio for the tower under consideration is found equal to: $r_{GMNIA} = 1,95$ [§2.4 (3)].
3. The calibration factor: k_{GMNIA} varies within the range: $0.8 < k_{GMNIA} < 1.2$ [EC3-1-6 §8.6 (26)]. Since no specific data is available, the most unfavorable value is adopted: $k_{GMNIA} = 0.80$.
4. Substituting the values, it is verified that the design buckling resistance ratio satisfies the condition:

$$r_{Rd} = \frac{k_{GMNIA} \cdot r_{GMNIA}}{\gamma_{m1}} = \frac{0,80 \cdot 1,95}{1,1} = 1,42 > 1,00$$

2.5.2. [EC3-1-6 §8.6]: Design by global numerical analysis using MNA and LBA analyses

1. The characteristic buckling resistance ratio for the perfect shell, is estimated by the [MNA] analysis: $r_{Rk} = 2.05$.
2. The overall buckling reduction factor $[\chi_{ov}]$ is a function of the parameters:

$\chi_{ov} = f(\lambda_{ov}, \lambda_{ov,0}, r_{ov}, \beta_{ov}, n_{ov})$, as determined in [EC 3-1-6 Annex D §1.2.2]:

$$\lambda_{ov,0} = 0,20 \quad \beta_{ov} = 0,60 \quad n_{ov} = 1,00$$

$$r_{ov} = a_x = \frac{0.62}{1 + (1.91/Q)^{1,44} \cdot (r/t)^{0,72}}$$

Introducing the appropriate values of the parameters, $[\chi]$ is obtained by the formulas of [EC 3-1-6 §8.5.2.2]: $\chi_{ov} = 0,71$

3. After substitution, it is verified that the design buckling resistance ratio satisfies the condition:

$$r_{Rd} = \frac{X_{ov} \cdot r_{Rk}}{Y_{m1}} = \frac{0,71 \cdot 2,05}{1,1} = 1,32 > 1,00$$

2.5.3. [EC3-1-6 §8.5]: Stress design

1. The buckling check of the shell according to the stress design method was attempted and the results of the calculations are presented in Table [T-2.5.3] (the combined check was not carried out).

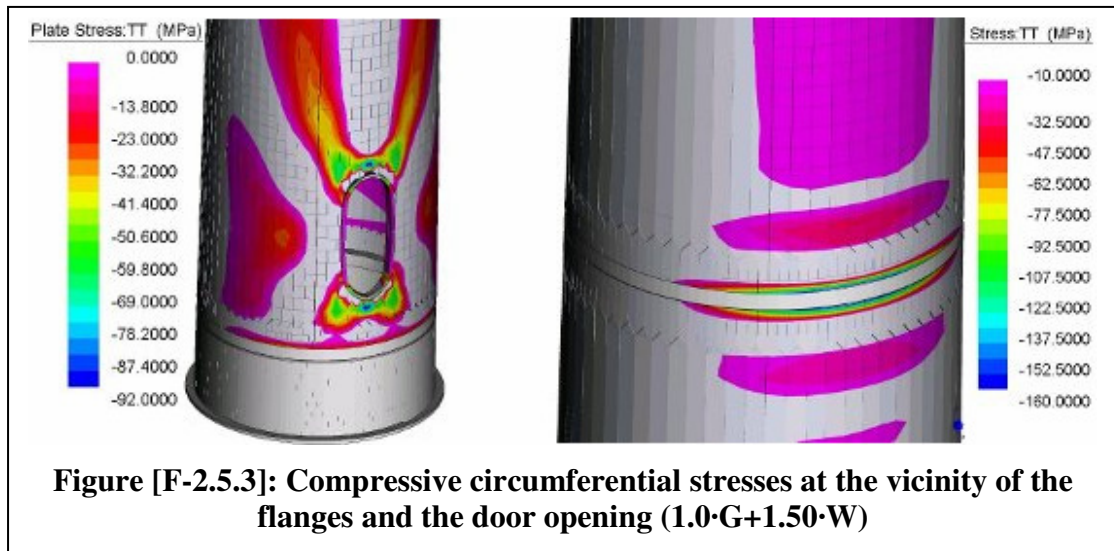
Table [T-2.5.3]: Stress design						
Courses	L	R _{mean}	T _{mean}	σ _{χ,Rd} / σ _{χ,Ed}	σ _{θ,Rd} / σ _{θ,Ed}	σ _{τ,Rd} / σ _{τ,Ed}
Lower	21.500	2.064	24,7	236 / 327	13,7 / 55,0	103 / 57,0
Middle	26.395	1.850	18,5	208 / 304	7,60 / 62,0	74,0 / 8,0
Upper	27.425	1.603	13,6	186 / 253	4,95 / 68,0	55,0 / 8,0

2. As reported in WP1.6 and illustrated in this specific case, this method leads inevitably to the installation of stiffening rings. Taking for example the relevant formulas (see [EC 3-1-6 Annex D]), in the case of long cylinders:

$$\sigma_{\theta,Rcr} = E \cdot \left(\frac{t}{r}\right)^2 \cdot \left[0,275 + 2,03 \cdot \left(\frac{C_{\theta}}{\lambda}\right)^4 \cdot \frac{t^6}{r^2} \right]$$

the critical circumferential buckling stress [σ_{θ,Rcr}] is diminished for a given significant magnitude of the length between boundaries (flanges, stiffening rings or base support), resulting thus in unacceptable shell plate thicknesses.

3. This noteworthy deviation of the stress design method from the previously mentioned global numerical analyses should be imputed to the fact that the buckling check along the boundaries is excluded only for the combined compression and not for the meridional, circumferential and shear components although, as demonstrated by the [LBA] buckling analysis, no eigenvector is reported there, even to the imperfect shell. Thus, the high intensity design stresses at the vicinity of the boundaries (see Figure [F-2.5.3]), combined with the restricted values of the buckling resistance stresses, when no intermediate stiffeners are used, results in low strength to design load ratios, of questionable reliability.



2.6. Concluding remarks

1. The results from the [LBA] analyses, considering elastic behavior of the structure for the perfect shell and the various types of imperfections, the influence of the imperfections, as described in [EC 3-1-6], is not significant to the buckling resistance of the tower.
2. On the other hand, the results from the limit load analyses indicate that the tower has been designed to reach the plastic limit load before approaching the bifurcation point. The collapse is induced by the shell buckling at the plasticized areas, where the plasticizing Von Mises stresses are components of the compressive meridional and circumferential stresses. It should be mentioned that fatigue limit state is not considered in the present analyses.
3. 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 [EC 3-1-6 §8.6]. In this case the imperfections are introduced indirectly, by the employment of the overall elastic imperfection factor $[r_{ROV}]$.
4. The global numerical analysis using [GMNIA] analysis design according to [EC 3-1-6 §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 $[k_{GMNIA}]$.
5. The stress design procedure of [EC 3-1-6 §8.5] results in rather conservative values, especially considering the circumferential stresses. The use of stiffening rings for this type of analysis is inevitable.

2.7. References

- [1] EN 1993-1-1: Design of steel structures – General rules and rules for buildings, 2005
- [2] EN 1993-3-1: 2006: Design of steel structures – Towers, masts and chimneys – Towers and masts, 2006
- [3] EC 3-1-6: Design of steel structures – Strength and stability of shell structures, 2006
- [4] GL Wind 2003 IV – Part1: Guideline for the Certification of Wind Turbines, 2004