STEEL WIND TURBINE TOWERS WITH INTERNAL RING STIFFENERS: ANALYSIS AND NUMERICAL INVESTIGATION

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ABSTRACT

Recent environmental phenomena along with the upcoming fossil fuel shortage make the use of renewable energy sources imperative in the near future. Wind energy, being the dominant clean energy source has matured throughout the years, aiming to expand further.To this end, advanced understanding of the wind turbine towers' structural detailing is critical for the improvement of their performance. This way, more efficient, durable and robust structures can be erected, facilitating their wider application and consequently achieving increase of wind energy production. The present study identifies the contribution of steel internal circumferential stiffening rings placed along the tower height to the overall structural behavior of the tower. The stiffening rings are suggested as a mean to reduce local buckling phenomena, increasing the buckling capacity of these slender steel structures. The beneficial role of the rings is established by the utilization of finite element model of the wind tower. Different distributions and locations of the rings are parametrically analyzed.

1 INTRODUCTION

Sustainable, alternative energy harvesting is becoming day-by-day indispensable considering the constantly deteriorating current climate conditions and the upcoming shortage of fossil fuels. The increasing energy demand globally, along with the need of eliminating the emissions produced by the traditional energy production procedures have led both Europe and the rest of the world to implement rules and regulations demanding up to 25% of total power used to come from sustainable means.Wind energy, being a powerfull sector of alternative energy generation, has been developed excessively in the past decades with great potential for future both on- and off-shore applications. Recently, addressing wind energy potential, taller wind energy structures with tower heights over 80m are introduced, the so called next generation wind turbines, aiming to take advantage the higher velocities that correspond at higher altitudes. In this framework, the tower component of the wind turbines plays significant role and improving understanding of its structural characteristics and behavioural aspects is imperative to provide innovative solutions in the tower design at these heights. It can result in more reliable and safer wind energy structures and subsequently, to wider application. The tower component doesn't just carry the other elemens of the wind turbine, i.e. the weight of the nacelle and the rotor blades, but it additionally absorbs the static loads caused by the alternating power of the wind. Wind turbine towers are manufactured by many materials, including steel, concrete and lately composites. The majority of all wind turbine towers are tubular tapered steel towers, i.e. they gradually narrow towards the top, and they are the most broadly used due to easier mounting and better structural detailing compared to the other two configurations. Significant parameteres that influence the decision for the required material and the construction costs are the height and the stiffness [1]. Due to transportational reasons, they are not manufactured as one full height long tower but are composed of a number of cylindrical shells, depending on the height, manufactured in the factory and mounted on site by means of bolted flanges with fully preloaded bolts.The structural configurations that have been implemented by now in this sector are the following three: a cylindrical tower with a circular cross-section, a jacket tower with a truss structure and a hybrid tower which is a combination of truss substructure and a cylindrical upper part.

With respect to their structural performance, there have been several studies focusing on the design parameters of the tower and its response under static and dynamic loadings. Lavassas et al. [3] have analyzed a prototype 1 MW steel wind turbine tower, Bazeos et al. [2] conducted a stability analysis on a steel wind turbine tower. Kang-Su Lee and Hyung-Joon Bang [4] have performed the numerical analysis of a collapsed wind turbine tower in Korea. Dimopoulos and Gantes [5] have performed experiments and have elaborated numerical results on different types of stiffening around the openings of wind turbine towers. Arasu et al. [6] and Nuta et al. [7] have performed seismic analysis on wind turbine steel towers. The structural optimization of the tower, which is the main supporting structure of a wind power plant, is considered rather important, since fatal accidents and great economic losses have been reported in wind farm accident reports [4], [5], [8] due to structural failure. The main failure mode for this type of long slender structures is local lateral buckling since the main loads occur in the lateral direction [2] and therefore limiting shell buckling and further stiffening it against local buckling with the implementation of ring stiffeners is of importance. Structural optimization of wind turbine tower has been the topic of research of Uys et al. [9] and Negm et al. [10].

There has been great scientific progress in the field of shell structures, but there is limited work conducted focusing on wind turbine towers'stiffening schemes, evaluating their impact on the structural response and their effectiveness against buckling phenomenon [11]. Within the framework of the study, the under combined loading is investigated and a numerical study is conducted in order to examine the effect of ring stiffeners against local buckling and thus improving the overall behavior of the structure. The circular stiffeners are placed at regular intervals along the height of the towers. The software used for the finite element analysis is the commercial finite element program ABAQUS [12]. From a structural point of view, the contribution of ring stiffeners agains local bucking is highlighted and their optimal number and position along the tower length is discussed. The unstiffened structure is also analysed at the beginning of the present study performing a linear perturbation analysis. The first eigenmode shape is then used in the non-linear analysis with imperfections as the initial imperfections of the structure. The same procedure is also followed for the different stiffening schemes investigated.

2 CYLINDRICAL SHELLS

Shell elements due to their geometry have the advantage of carrying great loads with small thicknesses[13].Therefore they are characterised by their wide application in multiple engineering branches including examples like silos, chimneys, nuclear reactors, pipelines etc. and recently also wind turbine towers.This kind of structures due to their small thicknesses compared to the other dimensions suffer from buckling phenomena.Over the past decades shell buckling has been thoroughly investigated both exeprimentally and numerically.There have been entire books devoted to investigate and interpret the behavior of shells like the ones written by Timoshenko [14], Bazant [15], Vinson [16], Galambos [17], Teng and Rotter [18]. From the very early experimental results, it was indicated that real cylinders buckle at loads significantly lower than the classical analytical predicted buckling load, which is the linear bifurcation load based on the assumptions of simple supports and a membrane state of prebuckling stress distribution. This discrepancy between experimental and theoretical results was attributed to initial geometrical imperfections, loading eccentricities, boundary conditions and difference in thicknesses and material properties. As the shell slenderness increases the influence of those factors becomes more important.

2.1 Wind turbine towers

The study of cylindrical wind turbine towers with circular cross-section lies in the field of cylindrical shells under combined loading of transverse and compressive loads and bending moment. This matter has been investigated by Witterstetter [19]. The issue of limiting shell buckling and further stiffening it against local buckling by means of circular stiffeners that are placed at regular intervals along the cylinder has been examined by Jumikis and Rotter [20], Rotter [21], Teng and Rotter [22], Lemak and Studnicka [11]. They are considered thin-walled structures as their thickness is much less than other structural dimensions. Thin shells structures transfer loading by means of the membrane tensional and compressive forces that act in the walls of the shell. Their efficiency in load bearing capacity is rather high under symmetrical loading and boundary conditions whereas unpredictable behavior is observed under asymmetrical loading or local load concentration. According to the complexity of the structure and desired level of analysis the new European Structural Code (EN 1993-1-6) [23] for the design of steel shell structures defines the different methods for defining the critical buckling stresses.

- for axisymmetric conditions only, the membrane theory can be used, using the stress analytical equations introduced in Annex D of the Eurocode [23],
- linear elastic analysis which is a minimum requirement for stress analysis under general loading conditions,
- linear elastic bifurcation analysis [LBA] which is required to obtain the critical buckling resistance,
- material nonlinear analysis [MNA], which is used for shells under general loading conditions to obtain the reference plastic resistance, and
- material and geometric imperfection analysis [GMNIA], using appropriate imperfections from the LBA and appropriate calibration factors.

2.2 Cylindrical shells buckling analysis

A first approach to perform the buckling analysis of a structure is to perform a linear bifurcation analysis to determine the elastic buckling load of the perfect structure. Shells and especially cylindrical shells have been proved to buckle in loads lower than their bifurcation load. Since this phenomenon is attributed to initial imperfections and boundary conditions of the structure, there is a method followed excessively accounting for buckling problems of shells. The procedure proposed by Speicher and Saal [24] is that the shape of the first eigenmode of the structure is introduced as the initial imperfections of the structure in the performance of the nonlinear analysis giving satisfactory results. The issue when following this approach is that the reduction factors applied to account for geometric imperfections and plasticity are difficult to determine, but general values for structures are included in the standards.
In the European Standard [23] the ultimate limit state of buckling is defined as the condition which all or part of the structure develops large displacements normal to the shell surface, caused by loss of stability and leading to inability to sustain any increase in the stress resultants. To investigate the contribution of stiffening rings to the buckling capacity of a wind turbine tower, geometrically and materially nonlinear analysis is followed as proposed both for the ultimate limit state and buckling limit state in the European Standard [23]. To further analyze the behavior of the structure against buckling, where the contribution of the stiffening rings is considered crucial a geometrically nonlinear imperfection analysis is performed coupled with a local bifurcation analysis (LBA) to calculate the appropriate imperfections. Special attention has to be drawn to the boundary conditions being relevant to the incremental displacements of buckling. Shell structures are very sensitive to the type of boundary conditions andfor the example of ring-stiffened cylindrical shells [23], the stiffening rings are considered a supplementary boundary conditioncharacterized as a pinned boundary condition, therefore constraining radially the shell while keeping the rotational and meridional degree of freedom, free. According to the Standards' treatment (Eurocode [23] and DNV [25]) consequent parts of the shell are considered as simply supported at both ends and therefore the effect of local buckling is limited and the structure has a deformed shape closer to the beam theory than without the implementation of the stiffening rings. This is proved also by the higher critical load of the stiffened structure compared to the unstiffened one. Both the European Standards and DNV cover design equations for either unstiffened shells that are pinned supported in both ends and ring stiffened cylindrical shells.

3 RING STIFFENERS' FUNCTION

Ring stiffeners are often used as an internal stiffening scheme of shell structures. As investigated and described by Bushnell, their commonest failure mode isout of plane buckling of the rings especially when they are attached to a shell intersection [26]. The minimum thickness of the shell wall and the critical values for their capacity are calculated according to the length to diameter ratio and the appropriate factor of safety. In cases of simply supported cylindrical shells but also in cases of built-in edges the internal constraints are not influencing greatly the critical load of the structure. Wind turbine towers stiffened by internal rings, are long and slender cylinders and when these rings are rigid enought to carry lateral loads without in plane deformations, the critical buckling length is considered to be the distance between consequent rings.
The stiffeners under investigation in the present work are circular local stiffening members that pass around the internal circumference of the shell revolution at a given point at the meridian. Their cross section varies in

different civil engineering applications from single plated to “T” or “L” profiles, but in the case of wind turbine towers where simplicity in implementation and less weldings is the target, single plated stiffening rings are the most common as shown in Fig.1 . These single plated rings when designed with appropriate thickness, their potential out of plane deformation is limited and their in-plane stiffness is comparably high.

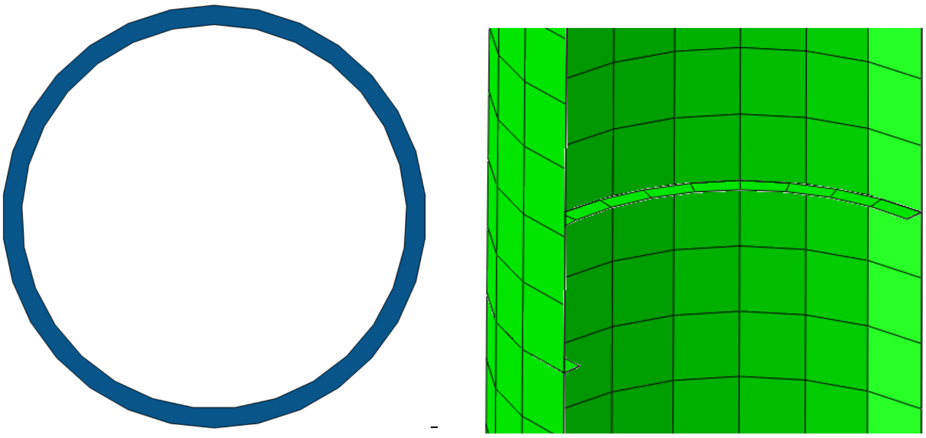

Figure 1. On the modeling of plated stiffening rings

In wind turbine tower design the beneficial impact of the implementation of stiffening rings can be observed in the design according to the plastic limit state and the buckling limit state as they are defined in the Eurocode. In the first, the beneficial influence of the stiffeners to the overall performance of the shell is mainly on the relief of the excessive strain concentration on the flanges due to the concentrated circumferential stresses. The stiffened tower response does not differ dramatically compared the unstiffened tower response when loaded identically. In the later, the contribution of the implementation of the stiffening rings can be observed in two cases. Firstly, investigating the unstiffened tower, the shell buckling strength is limited in the circumferential direction due to small thickness-to-radius ratio. With the implementation of the stiffening rings, the tower’s buckling strength is increased. Secondly, as described analytically in the Eurocode, the wind force around the circumference of the shell results in high circumferential stresses and second order effects as it tends to ovalize the circular cross-section. The presence of the circular stiffeners seems to retain the circular shape of the cross-section and the maximum concentrated stresses are noticeably relieved. For the performance of buckling assessment of shells, various methods are proposed in the Eurocode [23]. The critical buckling stresses can be determined using the global numerical material non-linear analysis (MNA) and local buckling analysis (LBA), the global numerical material non-linear imperfection analyses (GMNIA) or the stress design method described in the same document. In this stress design method introduced in Eurocode [23], there is an analytical equation provided (EN 1993-1-6 ANNEX D) where for long cylinders the critical circumferential buckling stress should be obtained from the following equations (1)-(3). In this framework, for long cylinders where

$$\frac{\omega}{C_\theta} > 1{,}63\frac{r}{t} \tag{1}$$

the critical circumferential buckling stress is

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

$$\omega = \frac{l}{\sqrt{rt}} \tag{3}$$

where: ω is the dimensionless length parameter, r the cylinder radius, t the shell thickness, l the cylinder length, Cθ the external pressure buckling factors according to boundary conditions and E the Young's modulus of the material. For long cylinders l is maximized and therefore the critical circumferential buckling stress σθ,Rcr is minimized. When implementing stiffening rings that retain the out of plane deformation of the shell, the critical buckling length is considered the length between boundaries or built-in edges. In this case, the smaller the l, the higher the circumferential buckling stresses. In DNV standards [25], the same statement is expresses although the calculation procedure differs. In these standards the characteristic buckling strength is calculated following formula (4).

$$f_E = C\frac{\pi^2 E}{12(1-\nu^2)}(\frac{t}{l})^2 \tag{4}$$

where, C is the reduced buckling coefficient which is independent of l, E is the Young's modulus, t is the shell thickness, ν is poisson's ratio and finally l is the distance between boundaries. In the case of ring stiffened cylinders it is stated that l is the distance between effective supports of the ring stiffened cylinder, which may be end closures, bulkheads and heavy ring frames. Therefore it is derived that well designed ring stiffeners, with the necessary thicknesses cut down the buckling length.

4 NUMERICAL ANALYSIS

4.1 Model Description

The wind turbine tower model used for conducting the analyses in the present study has a hub height of 76.15 meters and due to transportation limitations and construction restrictions, consists of 3 parts that are mounted on site. The three tower parts vary in length as it is mentioned: 21.8 m, 26.6 m and 27.8 m from bottom to top. The stiffening rings of the stiffened structures are implemented in equal numbers in each part with varying distances between them. In addition, the tower is not purely cylindrical as the lower diameter of the tower is 4.3 m and the top one is 3 m. The thickness of the shell wall is also varying along the tower height, starting from 12mm at the top to 30mm at the bottom. Since the focus is driven on local buckling phenomena, the imperfections of the tower wall have to be taken into account. Therefore the simulation of the tower was chosen to be conducted with the use of shell elements, since beam elements would not permit the calculation of the imperfections around the wall circumference. Solid elements were not chosen in order to save computational time and capacity. All the elements of the tower, both the outer wall and the internal rings are modeled with shell elements of type S4R as described in the Abaqus Manual [12] and are displayed in Fig 1. For the shell elements, in comparison to the continuum elements where the full 3-D geometry is user defined, only the conventional shell model geometry is designed and the shell thickness is taken into account by the section properties. Therefore, shell elements have both translational and rotational degrees of freedom at its nodes, whereas continuum elements provide only translational degrees of freedom at their nodes. The middle surface is considered to be the reference surface of the shell.

4.2 Loading Conditions

The loads that have to be taken into account when designing wind turbine towers are mainly the self-weight of the tower, the forces acting on the tower due to the function of the nacelle provided by the manufacturer, the fatigue loads that are again provided by the manufacturer, possible icing around the shell of the tower and the wind load that is distributed along the height of the tower.

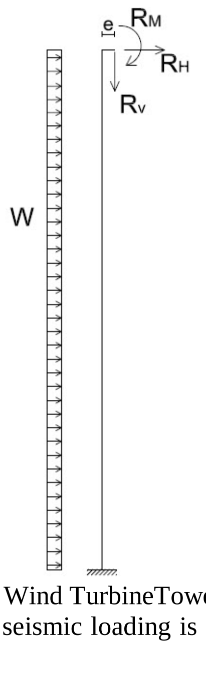

Figure 2. Wind TurbineTower Loads

In some areas of medium to high seismic risk, seismic loading is also taken into account in the design process.

For the analyses of the present study, focus is on the contribution of stiffening rings for improving the tower behavior against local buckling,therefore the fatigue, the seismic and icing loading are not taken into account. The self-weight of the tower's shell wall is automatically calculated introducing density in the material properties and calculating the geometrical date from the user designed shell dimensions and properties. The external forces that are taken into account in the analyses of the present study are shown in Fig.2.
The wind loading is a trapezoidal shaped distribution along the height and its distribution around the circular circumference is described by a complex formula. For the wind loading in the present case (W), a slightly simplified trapezoidal shape pressure is used for the implementation of the wind loading along the tower height and the wind distribution around the circumference is simulated in zones of 30 degrees with uniform pressure, giving similar results to the distribution proposed by the Eurocode [23]. The loads of the nacelle are provided by the producer and is as following: Total weight of nacelle (R_V), blades and rotor is positioned at the top having the center of gravity shifted horizontally +0.725 m from the axis of the tower and vertically +0.50 m above the upper flange level (+76.15 m). The rotation of the blades is producing according to manufacturer's data a concentrated horizontal force (R_H)at (76.15 + 0.5) m and a significant bending moment (R_M) at the same point. As formed the loading situation of the tower is the combination of the loads described above:

$$F = \{R_V + R_H + R_M\} + W \tag{5}$$

4.3. Analysis

Since the manufacturer's loads are applied eccentrically to the tower mid axis, in the finite element software the concentrated loads at the top of the tower are applied at a reference point ,taking into account the eccentricity of the rotor position, that is coupled to the top tower circumference. The gravity loads are generated automatically through the material density introduced and the shell wall geometry, while the wind loading is calculated at the model as pressure around the circumference of the cylindrical shell in zones as it is described in paragraph 4.2. The method chosen for the tower analysis is Geometrical and Material Nonlinear Imperfection Analysis and as proposed by Speicher and Saal [24] the equivalent imperfection is of the same form as the first bifurcation mode with an adjusted amplitude. Consequently a bifurcation analysis is first carried out in order to obtain the buckling shapes of the structure and the eigenvalues necessary for the non-linear static analysis. A comparison of the buckling shapes of the unstiffened structure and the various stiffened ones can be made from this step comparing the buckling shapes and the critical load calculated for the different structures. 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 355MPa and ultimate strength 510MPa. For the introduction of material plasticity date in Abaqus, the material properties have to be described in terms of plastic true stress and plastic true strain. In order to calculate the true material data, the following equations need to be used

$$\begin{aligned} \varepsilon_{true} &= \ln(1+\varepsilon_{nom}) \\ \sigma_{true} &= \sigma_{nom}(1+\varepsilon_{nom}) \\ \varepsilon_{true}^{pl} &= \varepsilon_{true} - \frac{\sigma_{true}}{E} \end{aligned} \tag{6}$$

After performing the bifurcation analysis and calculating the initial imperfections of the structure, material plasticity date is introduced in the numerical model and GMNIA analysis is performed. The steps and the results of the various analyses carried out are presented below.

5 RESULTS

The structures that are under investigation in the present work are firstly the unstiffened tower of 76,15m height and the same tower with the addition of ring stiffeners of equal number at each tower subpart. The cases analyzed are the tower with 1 ring at each part named 3R, with 3 rings at each part named 9R, with 5 rings at each part named 15R and with 7 rings at each part named 21R. For all the models GMNIA is implemented with the use of the 1st eigenmode shape of the structure as initial imperfections and the appropriate imperfection factor.

5.1. Eigenvalue analysis

For all the analyzed schemes, the mesh of the models is maintained the same to avoid any numerical incoherence. The only difference between the unstiffened structure and the stiffened ones is the introduction of internal stiffening rings. The unstiffened structure contains the flanged connections between two consequent tower parts that act also as stiffening rings. For the stiffened structures the rings are added at equal intervals along the tower subpart.

Table 1. Eigenvalue and GMNIA results

	1^{st} Eigenvalue	Difference	Ultimate Load	Difference
Tower without internal rings	**2,0056**	-	0,42 F	-
Tower with 3 internal rings	2,0326	1,35%	0,47 F	11,9%
Tower with 9 internal rings	2,0814	3,5%	0,49 F	16,67%
Tower with 15 internal rings	2,3316	16,58%	0,56 F	33,33%
Tower with 21 internal rings	2,3832	18,83%	0,57F	35,7%

In table one the beneficial impact of the introduction of the stiffening rings is shown. The critical buckling load is calculated for all the cases and it can be derived that the more the stiffeners, the higher the buckling load.

5.2. Static Nonlinear Analysis with Imperfections

The introduction of the rings is proved to be beneficial again in the Geometrical and Material Nonlinear Imperfection Analsis.

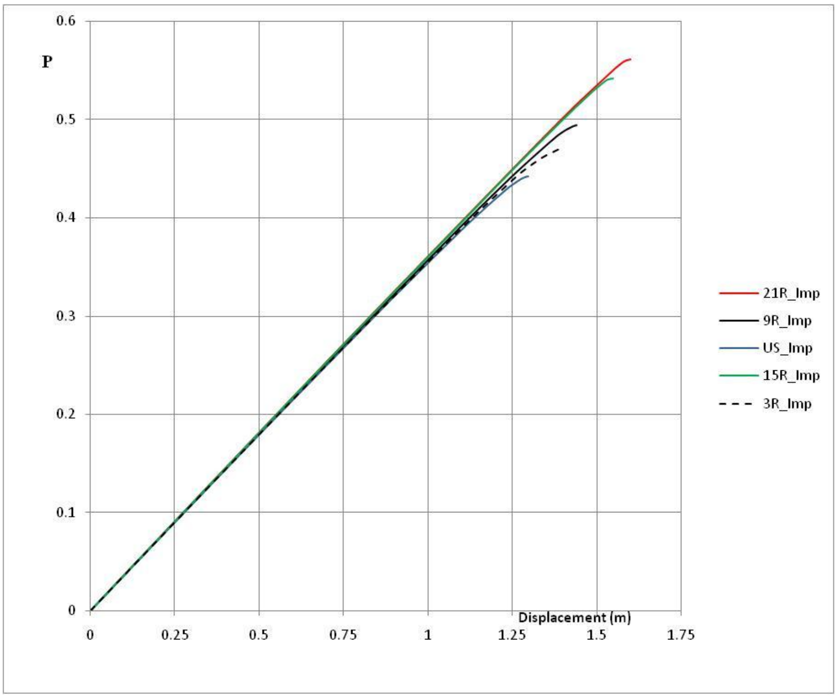

Figure 2. GMNIA analysis results

From Fig. 2 it can be observed that the more rings are introduced to the structure the higher the ultimate load is. In Table 1 the ultimate loads for all cases are presented in numbers. The difference in the tower response is calculated as the difference percentage comparing each stiffened case with the unstiffened one. Between cases 9R_Imp and 15R_Imp there is a remarkable increase in the ultimate load of the structures. This shows the importance of the rings position, since for the 15R_Imp case the 5 rings that are added to the upper tower part are all positioned in the area proved to be more vulnerable in the eigenvalue analysis. Between cases 15R_Imp and 21R_Imp no great difference is observed since the two additional rings in the upper part are put close to the tower part edges where the local buckling issue is not predicted to be intense.

6 CONCLUSIONS

The eigenvalue analysis shows the shape of initial imperfections, where the middle of the upper tower part is proved to buckle first, therefore this is the position with the higher value of initial imperfections. The eigenvalue and the material nonlinear imperfection analysis shows the influence of the imperfections to the ultimate limit load. As expected, the ultimate capacity of the structure with the implementation of initial imperfections appears much smaller than the elastic buckling load calculated in the eigenvalue analysis. The more dense the stiffening rings scheme the higher the tower ultimate capacity calculated, which is proved by the higher ultimate capacity for the models with increased number of implemented stiffeners. Finally, the positioning of rings across the tower height is important as stiffened models with rings added close to the area where failure is likely to occur show great difference compared to the other models where rings are positioned in less vulnerable areas. For the structure with five rings implemented in each tower part, a great increase in the ultimate capacity is observed due to the fact that all the stiffeners are positioned in the middle of the upper tower part, which is the position most likely to fail.

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