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FREE VIBRATION AND EARTHQUAKE BEHAVIOR OF SOLAR POWER PLANT CHIMNEYS

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Abstract. *In this paper, the basic mechanical principles for structural modeling of shells of revolution using ring elements are briefly summarized. For the representation of mechanical variables in hoop direction, Fourier series are used within this numerical model. Further, solution properties for free vibration and for transient dynamic problems are presented with special regard to ring element models. The presented algorithms are implemented in computer code ROSHE [7] which has been successfully applied to the analysis and design of a large number of cooling towers world wide. In this contribution, the challenging step from natural draft cooling towers which are currently built with standard heights up to 200 m towards large solar upwind power plant chimneys with heights of approximately 1000 m is dared. Hereby, the pre-design of a 1000 m solar chimney by Krätzig is taken as reference geometry. With this design, the dynamic behavior of this large 1000 m solar chimney subjected to horizontal earthquake acceleration is studied. The behavior of this tower is compared to dynamic behavior of cooling towers subjected to horizontal earthquake acceleration. While significant non-linear effects such as uplift from ground, cracking of columns and shell arise in cooling towers, the behavior of solar chimneys is totally different. Due to very high period of the first relevant eigenmotion, these structures behave similar to base isolated structures. The structure will not follow the ground motion in upper parts. Despite of the large height, uplift or loss of stability due to tilting will not become a problem. Tensile meridian membrane forces occur only in upper parts of the shell between 600 m and 1000 m. In this paper, the transient response of the 1000 m solar chimney is studied in comparison to a standard natural draft cooling tower of height 180 m. They are both subjected to NS component of Kern County earthquake acceleration with peak ground acceleration 0.35 g. Simplified response spectrum analysis with presentation of modal contributions is further conducted for a simplified beam model of the solar power plant chimney which confirms the obtained results. Powerful post processing tools have been developed and will be utilized within the presentation to visualize the structural response of both systems.*

1 INTRODUCTION / SUMMARY

Natural draft cooling towers up to heights 200 m are commonly found at every power plant where process water has to be cooled down. Due to their height and their slender wall thickness, they represent one of the most challenging and fascination civil engineering structure. The highest natural draft cooling tower world wide has been built at the Niederaussem power plant site close to Cologne in Germany. Cooling towers have to resist mainly dead weight, wind and earthquake action.

Recently, the discussion about solar power plant chimneys has come up again. Solar power plant chimneys use a huge circular glass roof collector to heat up the air. The air is flowing under this glass roof towards a high chimney in the middle. Due to the natural updraft effect through the chimney, the air flow is used to run several turbines which are installed at the bottom of the chimney. With such a solar chimney of height 1000 m and a collector roof of diameter 3 km, an electrical power output of 200 MWe can be generated.

For such a high structure, it must be questioned how it will behave when subjected to horizontal earthquake acceleration. Will earthquake be the dominating action? Will tilting become a problem? Which non-linear effects will occur?

For this clarification, a first step will be done within this paper. Therefore, a comparison of the structural behavior between natural draft cooling towers and solar power plant chimneys will be conducted which are both very slender shells of revolution with negative curvature.

A very simple but effective ring element model is used for this aim. Both structures are subjected to a representing earthquake acceleration (Kern County NS component) with maximum peak ground acceleration 3.50 m/s². The results for the solar power plant chimney are further confirmed by simple beam response spectrum analysis. It can be stated that tilting will not occur for the solar power plant chimney and significant tensile meridional stresses arise only in upper regions of the structure due to higher mode contribution.

2 MECHANICAL BACKGROUND AND FINITE ELEMENT CONCEPT

In Figure 1, a cut-out of a shell of revolution with its body forces and stress resultants is depicted. Internal stress resultants are described by membrane forces $n^{\alpha\beta}$ and bending moments $m^{\alpha\beta}$ which are obtained by numerical stress integration through section depth in general. They are obtained using Eq. (1) and Eq. (2) for arbitrary constitutive behavior [2].

$$n^{\alpha\beta} = \int_{-h/2}^{+h/2} \mu(\theta^3) \cdot [\delta_\lambda^\beta - \theta^3 \cdot b_\lambda^\beta] \cdot \sigma^{\alpha\lambda}(\theta^3)\, d\theta^3 \tag{1}$$

$$m^{\alpha\beta} = \int_{-h/2}^{+h/2} \mu(\theta^3) \cdot \theta^3 \cdot \sigma^{\alpha\beta}(\theta^3)\, d\theta^3 \tag{2}$$

Equations of motion are formulated using dynamic equilibrium by application of principle of virtual work with additional consideration of damping and inertia forces.

$$\begin{aligned} &\iint_A p^\alpha\, \delta u_\alpha + p^3\, \delta u_3\, dA + \oint_C n^\alpha\, \delta u_\alpha + n^3\, \delta u_3\, dC - \iint_A \tilde{n}^{\alpha\beta}\, \delta\alpha_{\alpha\beta} + m^{\alpha\beta}\, \delta\beta_{\alpha\beta}\, dA \\ &- \iint_A \rho h \cdot [a^{\alpha\beta}\, \ddot{u}_\alpha\, \delta u_\beta + \ddot{u}_3\, \delta u_3]\, dA - \iint_A [c_t\, a^{\alpha\beta}\, \dot{u}_\alpha\, \delta u_\beta + c_n\, \dot{u}_3\, \delta u_3]\, dA = 0 \end{aligned} \tag{3}$$

The principle of virtual work implies dynamic equilibrium in weak formulation. The equations of motion are solved by structural discretisation using finite ring elements. Further, the equations of motion are linearized to apply an implicit time integration scheme with corrective iteration steps [7], [8].

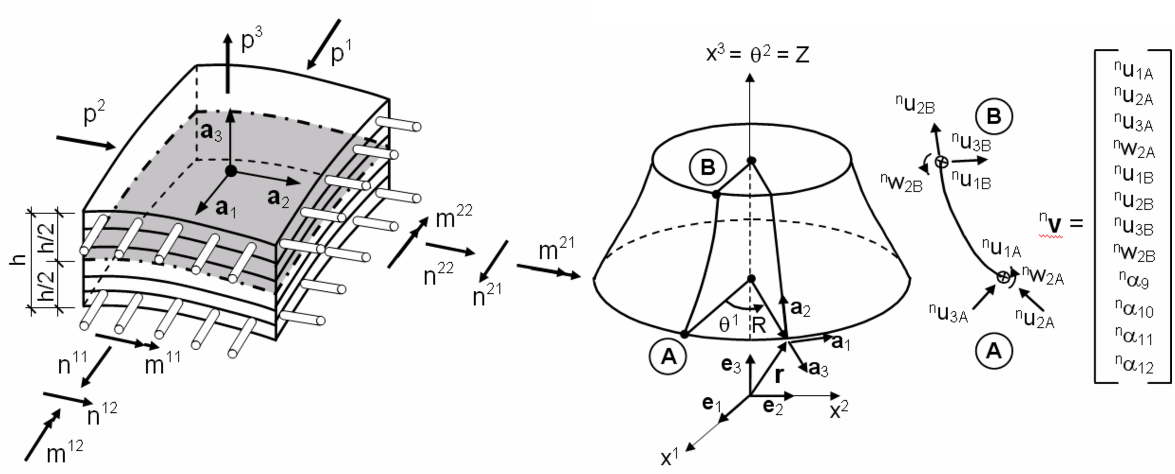

Figure 1: Numerical model for structural analysis of shells of revolution acc. to [7], [8], [9].

For structural analysis of shells of revolution, ring elements according to Figure 1 can be advantageously deployed since the computational effort is significantly decreased. The shell is described by its meridian curve. Each element possesses two nodes. Displacements are interpolated using cubic polynomials in meridian direction and a Fourier series in hoop direction. For linear elastic behavior, the structural problem is decoupled in Fourier terms: Body forces result in structural response in the same Fourier term, solely. This property is of special interest in linear earthquake analysis of shells of revolution which is conducted within this paper.

3 NUMERICAL MODELS FOR NDCT AND SPPC

In Figure 2, structural models of a modern natural draft cooling tower which has been recently built and of the pre-design of a 1000 m solar chimney by Krätzig [5], [6] are depicted.

The natural draft cooling tower consists of one upper ring beam which is modeled by a ring beam element. The double curved shell consists of 43 ring elements with a minimum wall thickness 20 cm in major parts of the shell. At its bottom, the shell is supported by V-truss columns (46 column pairs with diameter 90 cm). These truss columns are modeled by a special column macro ring element where 3D beams are coupled along the upper and lower nodal circles. The ring foundation is modeled by a ring beam element which is supported by a ring spring element to account for the elastic soil support. The geometry of the double curved middle surface of the shell is described by two hyperbolic functions below and above the throat of the shell which follows Eq. (4).

$$R(Z) = R_0 + a \cdot \sqrt{1 + \left(\frac{Z - Z_0}{b}\right)^2} \tag{4}$$

with R_0 = -15.3644	Z_0 = 115.83	a = 51.9644	b = 113.9896	below throat
R_0 = +36.3422	Z_0 = 115.83	a = 0.25780	b = 8.0293	above throat

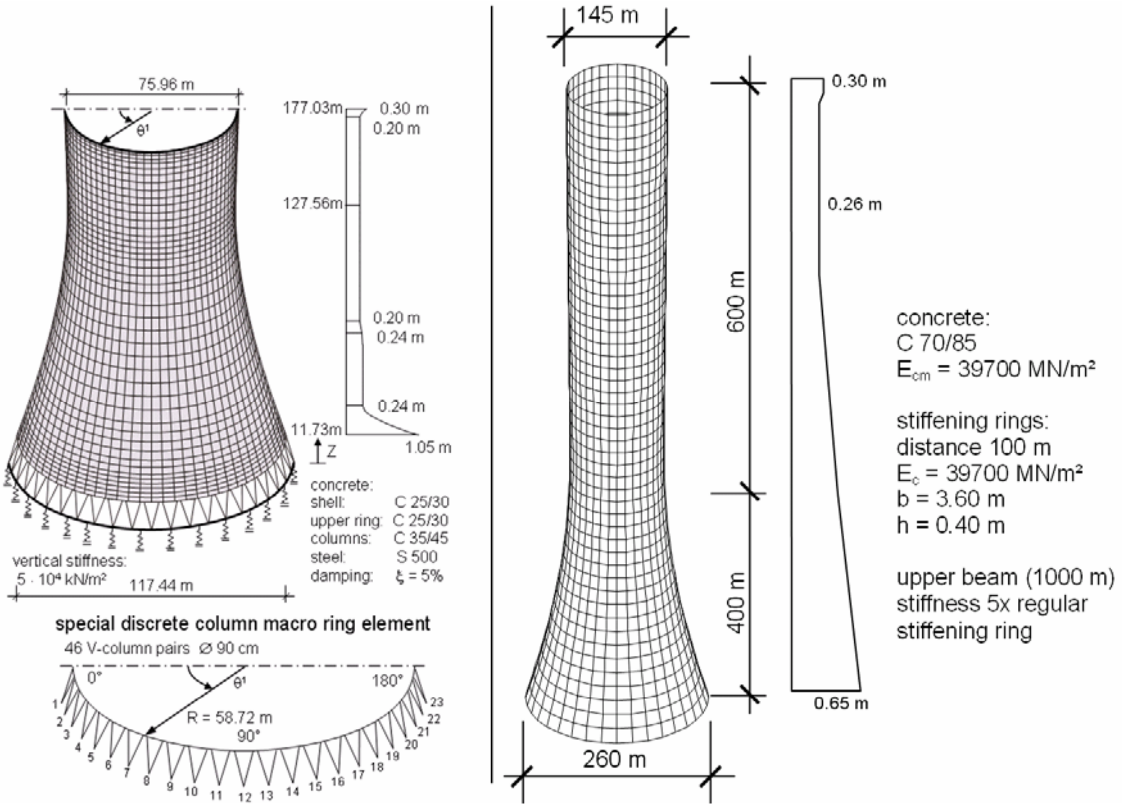

Figure 2: Structural models for earthquake analysis of NDCT and SPPC.

Further, the structural model of the 1000 m solar chimney can be found in Figure 2. This model consists of 50 shell ring elements while each element is representing a vertical section of 20 m. The base of the solar chimney is fixed. Modeling of air openings in the lower part is neglected for this study of global earthquake response behavior. The wall thickness of this 1000 m solar tower is starting at 65 cm at its bottom and reaches a minimum thickness of 26 cm in upper parts according to buckling safety requirements. At distances of 100 m, stiffening rings are attached to the shell. On top of the shell, a ring beam is attached with 5 times stiffness of standard stiffeners. A high performance concrete C 70/85 is used. All geometric and material properties are chosen according to pre-design by Krätzig, see [5], [6].

The structure is described by a double curved shell middle surface comparable to the presented natural draft cooling tower. The throat position is located at height 400 m. The geometry follows Eq. (4) with following parameters:

$R_0 = -8209.2709$	$Z_0 = 400.00$	$a = 8275.9709$	$b = 3227.9274$	below throat
$R_0 = +66.577380$	$Z_0 = 400.00$	$a = 0.1226204$	$b = 12.424908$	above throat

Although both structures are described in a similar way by a double curved shell surface (hyperbolic shape, negative curvature) with two geometric functions (below and above the throat) as can be seen in Figure 2, the earthquake behavior and structural response will be totally different due to significant discrepancies in mass and stiffness. The solar chimney will behave more or less like a very flexible cantilever beam similar to base isolated structures which can be already guessed.

4 DESCRIPTION OF EARTHQUAKE ACTION AND EQUIVALENT BODY FORCES

As reference earthquake for earthquake studies, Kern County earthquake NS component is used (21.07.1952, Taft Lincoln School, USGS Station 1095) [10]. This acceleration time history is depicted in Figure 3 with normalization to unit acceleration value 1 m/s².

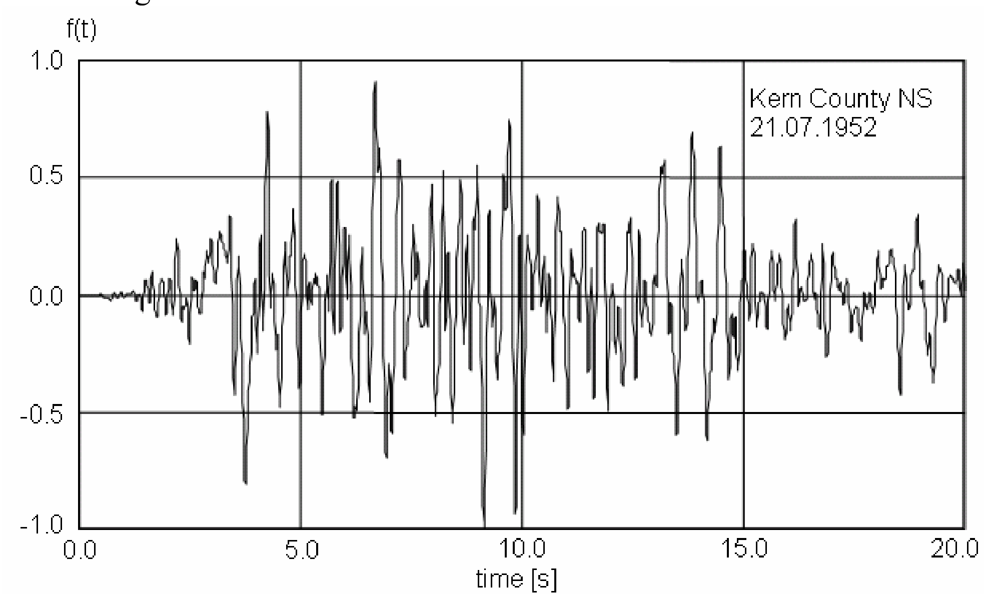

Figure 3: Acceleration time history of Kern County earthquake NS component.

Earthquake accelerations acting on a shell of revolution are described by the following body forces in tangential and horizontal direction with a_{gr} maximum ground acceleration, f(t) time history function (scaled to unit value), h shell thickness, θ^1 angle of circumference.

$p^{<1>} = +a_{gr} \cdot f(t) \cdot h \cdot \sin(\theta^1)$ tangential direction
$p^{<2>} = +a_{gr} \cdot f(t) \cdot h \cdot \cos(\theta^1) \cdot \sin(\alpha)$ meridional direction
$p^{<3>} = -a_{gr} \cdot f(t) \cdot h \cdot \cos(\theta^1) \cdot \cos(\alpha)$ normal direction
α inclination angle of meridian curve
$\tan \alpha = - dR / dZ$

Earthquake acceleration generates equivalent body forces in Fourier term n = 1, which implies circumferential distribution $\cos(n \cdot \theta^1)$ or $\sin(n \cdot \theta^1)$. Thus, for computation of linear structural earthquake response only Fourier term n = 1 is of interest so that the equations of motion can be simplified significantly.

5 EARTHQUAKE RESPONSE OF NDCT AND SPPC

5.1 Natural Eigenperiods

In Figure 4, the corresponding response spectrum for Kern County time history is presented scaled to unit ground acceleration (rigid body value at T = 0 s). Further, the response spectra according to DIN 4149 [4] for soil class A-R and C-S are presented for comparison purpose. By solving the linear eigenvalue problem of free vibration for Fourier term n = 1

$$({}^{11}\mathbf{K} - \omega^2 \cdot {}^{11}\mathbf{M}) \cdot \psi = \mathbf{0} \qquad (5)$$

the governing eigenperiods and eigenmodes of both systems are obtained. The following eigenmodes can be identified:

NDCT	1st mode n = 1	motion on elastic soil support	T = 0.73 s
NDCT	2nd mode n = 1	motion on elastic column support	T = 0.40 s
NDCT	3rd mode n = 1	shell bending mode	T = 0.18 s
SPPC	1st mode n = 1	cantilever bending mode #1	T = 5.66 s
SPPC	2nd mode n = 1	cantilever bending mode #2	T = 1.36 s
SPPC	3rd mode n = 1	cantilever bending mode #3	T = 0.66 s

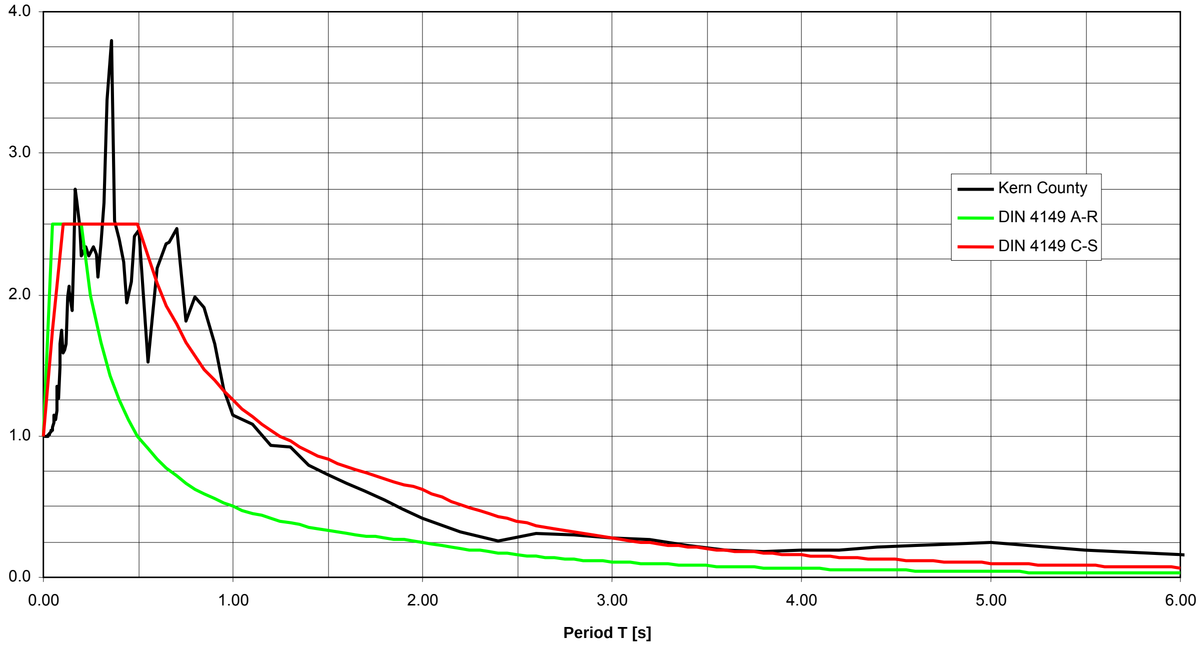

Figure 4: Eigenperiods of NDCT and SPPC with corresponding response spectra.

From natural periods, it becomes obvious that the SPPC in comparison to the NDCT is a very soft structure. The 1st period of SPPC is almost 10 times higher than the 1st period of NDCT. The SPPC is too flexible to follow the ground motion. In contrast, the NDCT is strongly affected by earthquake acceleration since the governing eigenperiods coincide with maximum response accelerations. This fact results in very high column stresses (in case of V-columns: compressive and tensile forces, in case of I-beams: bending moments).

5.2 Transient Response to Kern County Earthquake

Linear transient time history analyses have been carried out for the two presented systems NDCT and SPPC. Hereby, Kern County NS acceleration according to Figure 3 with maximum peak acceleration 0.35 g has been used as reference.

In general, the following effects can be observed for NDCT subjected to earthquake acceleration:

- uplift from ground (in case of raft foundation)
- cracking of concrete of supporting columns (exceedance of concrete tensile strength)
- cracking of shell (exceedance of concrete tensile strength)

From conducted analyses for SPPC subjected to horizontal earthquake acceleration, the following effects can be observed:

- Uplift from ground does not occur. Tilting will not become a problem since 1st period is so high that the upper structure cannot follow the ground acceleration.
- Tensile meridional forces arise only in upper parts of the structure between 600 m – 1000 m.
- Higher modes which describe higher bending modes of the cantilever system will become relevant for the structural response which can be seen from Figure 5.

In Figure 5, the structural response in terms of maximum meridional forces is depicted for both systems. For the NDCT, it can be observed that under dead weight, the complete shell is under meridional compression. Horizontal earthquake acceleration generates tensile meridional forces throughout most parts of the shell. Cracking will occur at meridional forces 0.20 m x 2.56 MN/m² = 510 kN/m. The prestressing effect due to dead weight is totally used up. For the SPPC system, it can be observed that dead weight still represents the dominating part. Tilting will not occur since the base is in compression. Assuming a tensile strength of HPC C 70/85 f_{ctm} = 4.60 MN/m², the cracking meridional force 0.26 m x 4.60 MN/m² = 1200 kN/m is not exceeded. Since the first period of the SPPC is such high, the structure cannot follow the base acceleration. Therefore, higher modes have to be considered to correctly asses the structural response in upper parts. The contribution of different vibration modes to the dynamic response can be found in Figure 5.

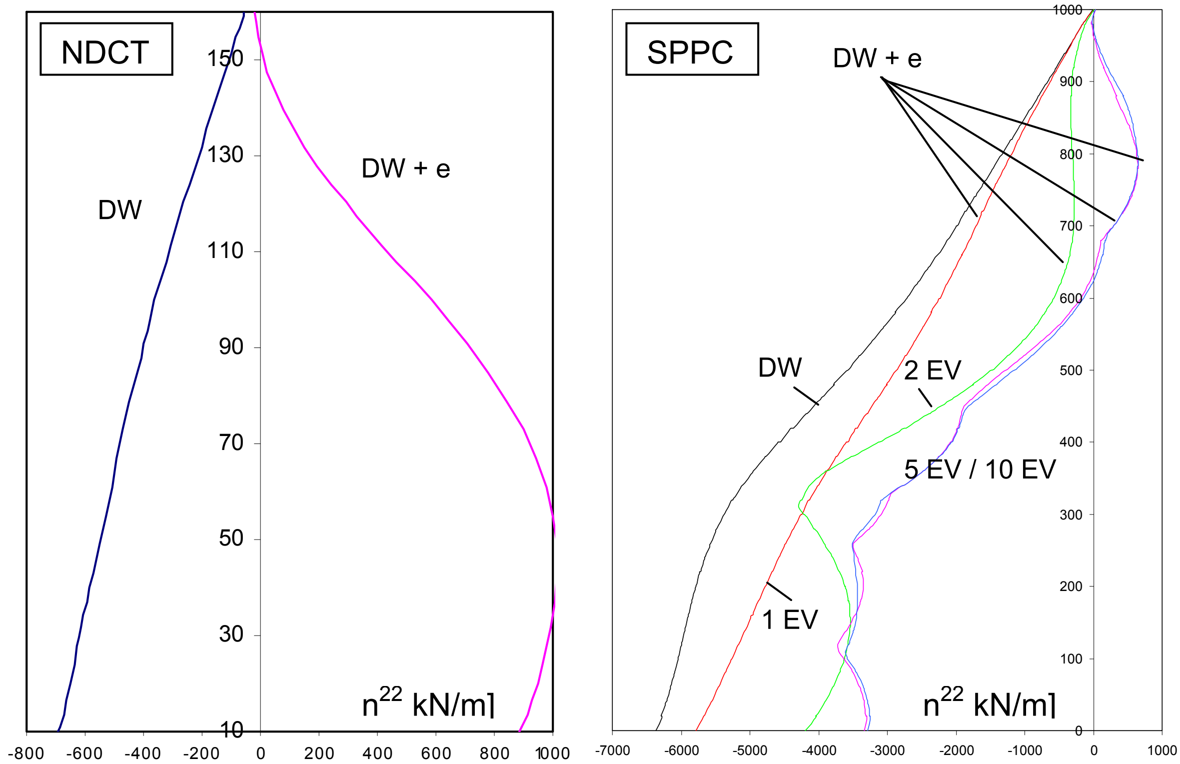

Figure 5: Meridional tensile forces due to earthquake acceleration Kern County NS component (0.35 g).

5.3 Simplified Analysis of SPPC Using Response Spectrum Method for Cantilever

The total mass of the solar power plant chimney yields approximately 501.000 t. The equation of motion for a structure subjected to earthquake motion is given in Eq. (6). By modal transformation, Eq. (7) is obtained. Hereby, the physical degrees of freedom **V** are transformed to modal coordinates q so that the system of coupled equations of motion is decoupled into independent equivalent single degree of freedom systems.

$$\boldsymbol{K} \cdot \boldsymbol{V} + \boldsymbol{C} \cdot \dot{\boldsymbol{V}} + \boldsymbol{M} \cdot \ddot{\boldsymbol{V}} = -\boldsymbol{M} \cdot \boldsymbol{I} \cdot a_{gr} \cdot f(t) \tag{6}$$

$$\boldsymbol{\psi}^{T} \cdot \boldsymbol{K} \cdot \boldsymbol{\psi} \cdot q + \boldsymbol{\psi}^{T} \cdot \boldsymbol{C} \cdot \boldsymbol{\psi} \cdot \dot{q} + \boldsymbol{\psi}^{T} \cdot \boldsymbol{M} \cdot \boldsymbol{\psi} \cdot \ddot{q} = -\boldsymbol{\psi}^{T} \cdot \boldsymbol{M} \cdot \boldsymbol{I} \cdot a_{gr} \cdot f(t) \tag{7}$$

By using the response spectrum diagram, the maximum response acceleration is obtained in dependence of the natural eigenperiod T of the considered eigenmode and in dependence of damping ratio according to Eq. (8). With this maximum response acceleration, equivalent static loads $\mathbf{H}_{E,i}$ for each eigenform i can be computed using Eq. (10).

$$\max \ddot{q}_i = \frac{\boldsymbol{\psi}_i^{T} \cdot \boldsymbol{M} \cdot \boldsymbol{I}}{\boldsymbol{\psi}_i^{T} \cdot \boldsymbol{M} \cdot \boldsymbol{\psi}_i} \cdot a_{gr} \cdot \beta(T, \xi_i) \tag{8}$$

$$H_{E,i} = \boldsymbol{M} \cdot \boldsymbol{\psi}_i \cdot \max \ddot{q}_i = \boldsymbol{M} \cdot \boldsymbol{\psi}_i \cdot \frac{\boldsymbol{\psi}_i^{T} \cdot \boldsymbol{M} \cdot \boldsymbol{I}}{\boldsymbol{\psi}_i^{T} \cdot \boldsymbol{M} \cdot \boldsymbol{\psi}_i} \cdot a_{gr} \cdot \beta(T, \xi_i) \tag{9}$$

The results from equivalent static loads for each eigenform now have to be combined again for the total solution. Results for two different methods are presented in the following: square root of sum of squares (SRSS) and sum of absolute sum.

The results of section 5.2 can be confirmed by using simplified cantilever beam analysis in conjunction with response spectrum analysis. Tilting and uplift from ground will not become a problem while tensile meridional forces arise only in the upper part of shell for the solar power plant chimney. Meridional forces using method SRSS and ABSSUM are depicted in Figure 6. ABSSUM rule provides more unfavorable results since single mode contributions are directly added assuming that maximum response for all contributing modes occurs at the same time.

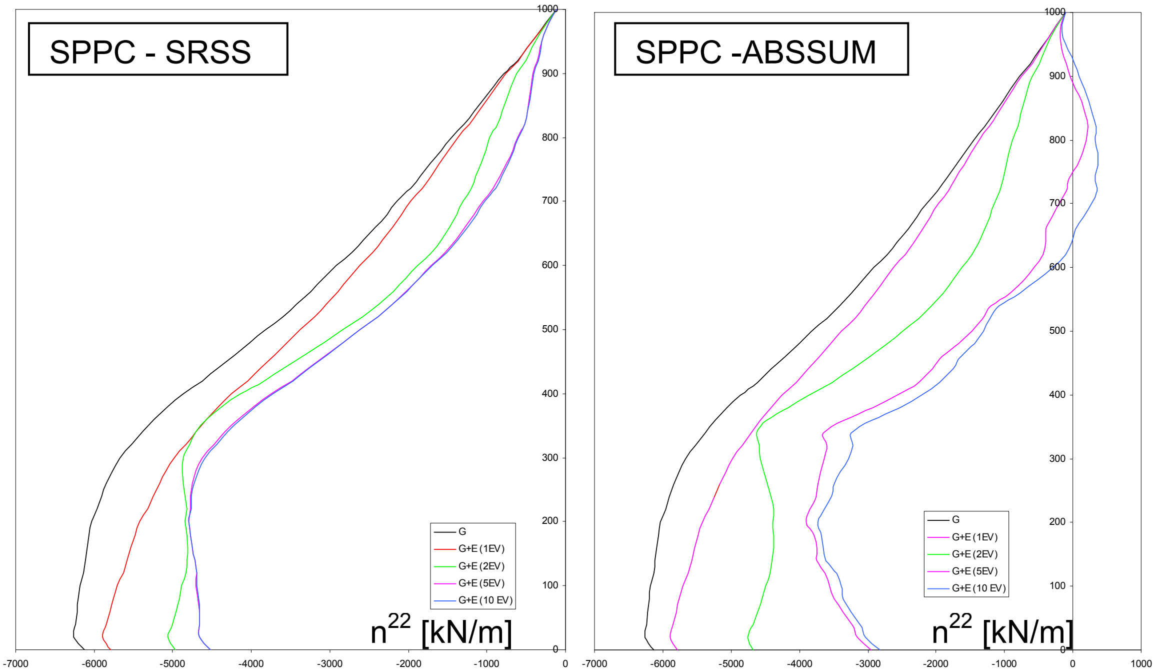

Figure 6: Simplified analysis using response spectrum analysis (SRSS and ABSSUM).

6 CONCLUSION

This contribution shall be only regarded as a first step into dynamic earthquake behavior of solar chimneys subjected to earthquake acceleration. Existing algorithms which have been developed for general shells of revolution and successfully applied to the design of a large number of natural draft cooling towers by the author world wide are deployed to this end. Although the structures SPPC and NDCT are described in a similar way by double curved hyperbolic shapes, the response to earthquake accelerations is totally different. Due to absolute values of eigenperiod, NDCTs behave more critical under earthquake acceleration than SPPCs. Non-linear effects such as uplift, cracking of shell and columns become significant for NDCT while for the analyzed 1000 m SPPC not even tilting will become a problem.

The results presented in this contribution have been obtained using transient time integration analysis for a representative earthquake acceleration time history for a full shell model (ring elements). Simplified response spectrum analysis method (hand calculation) using a cantilever beam model has been used for checking purpose which confirms the obtained results.

However, this study can be only seen as a first approach. Only a base acceleration by a rigid body ground motion has been considered (base of structure remains in original shape). Further effort has to be spent into design studies and into research in this new and demanding field of SPPC structures.

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