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Avoiding retroactive effects caused by the demolishing by blasting of the cooling towers of Philippsburg power plant – analysis and results of the verification process

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Abstract

In the course of the resolution from 2002 for the withdrawal from nuclear energy and further resolutions of the Bundestag in connection with the events in Fukushima in 2011, the two reactor blocks of the Philippsburg nuclear power plant were shut down. The first block (KKP1) went offline on March 17, 2011. The second block (KKP2) was shut down on December 31, 2019. In the future, a converter system for electricity from renewable sources will be built on the plant. To ensure the future conversion of the facility, the two cooling towers were dismantled by blasting.

The State Ministry for the Environment, Climate Protection and the Energy Sector of Baden-Württemberg requested proof that the demolition was free of retroactive effects. During this verification process, it had also to be taken into account that at the planned time of the blasting active fuel elements were still stored and cooled in the plant. The two towers were demolished on May 14, 2020. The blast was time staggered in order to avoid vibrations, which were induced into the soil by the collapse of the towers from being superimposed. The demolition was accompanied by measurements of vibration velocities at representative locations (open field and building) which proved that in advance determined limits were not surpassed.

This paper summarizes the verification process that ensures that there were no retroactive effects with regard to the vibrations caused by the collapse of the cooling towers. First, the definition of the desired level of security is discussed. The verification strategy is then examined. Particular attention is paid to the determination of reliable vibration predictions and numerical simulations of the structural behavior of the briefly remaining tower, which had to withstand the vibrations caused by the collapse of the first tower.

1. Introduction

In the course of the resolution from 2002 for the withdrawal from nuclear energy and further resolutions of the Bundestag in connection with the events in Fukushima in 2011, the two reactor blocks of the Philippsburg nuclear power plant were shut down. The first block (KKP1) went offline on March 17, 2011. The second block (KKP2) was shut down on December 31, 2019. In the future, a converter system for electricity from renewable sources will be built on the plant. To ensure the future conversion of the facility, the two cooling towers were dismantled by blasting. Figure 1 shows an overview on the power-plant, the cooling towers and the predicted area of debris after the dismantling by blasting. It is worth noting that the areas have a certain overlapping. The tower marked with “1” had to be dismantled first to avoid a collision of the towers.

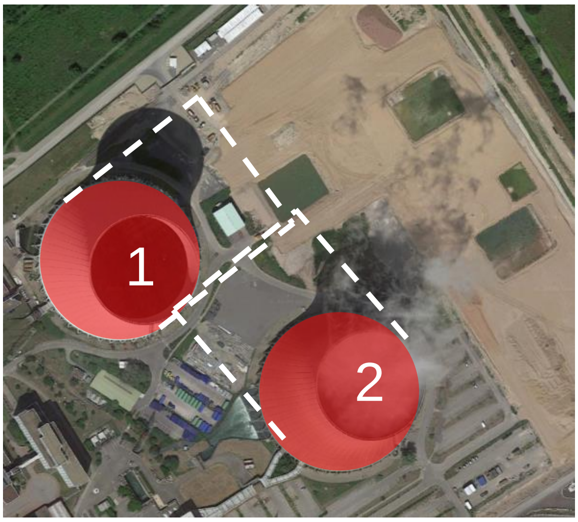

Figure 1: Left: Overview on Philippsburg powerplant – Right: The two cooling towers and predicted debris area after dismantling by blasting

The main data concerning construction details, dimensions and mass of one of the Philippsburg hyperbolic cooling towers is shown in Figure 2 (both towers identical).

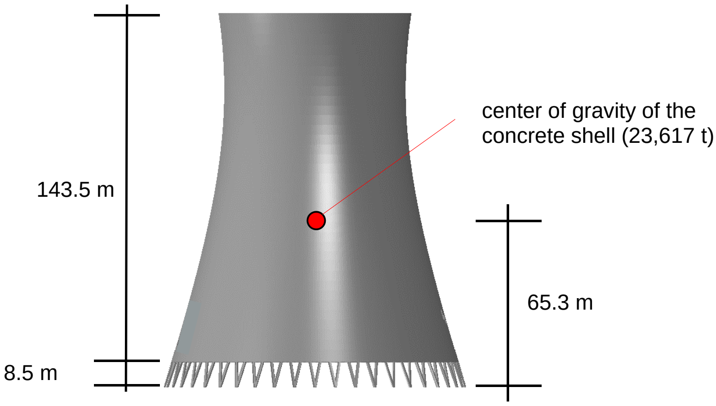

Figure 2: Philippsburg hyperbolic cooling tower and dimensions (rendering from finite element model)

Two methods were discussed for dismantling the cooling towers: conventional demolition and dismantling by blasting. The operating company chose the blast variant mainly because of its comparatively short duration. Figure 3 shows the dismantling concept.

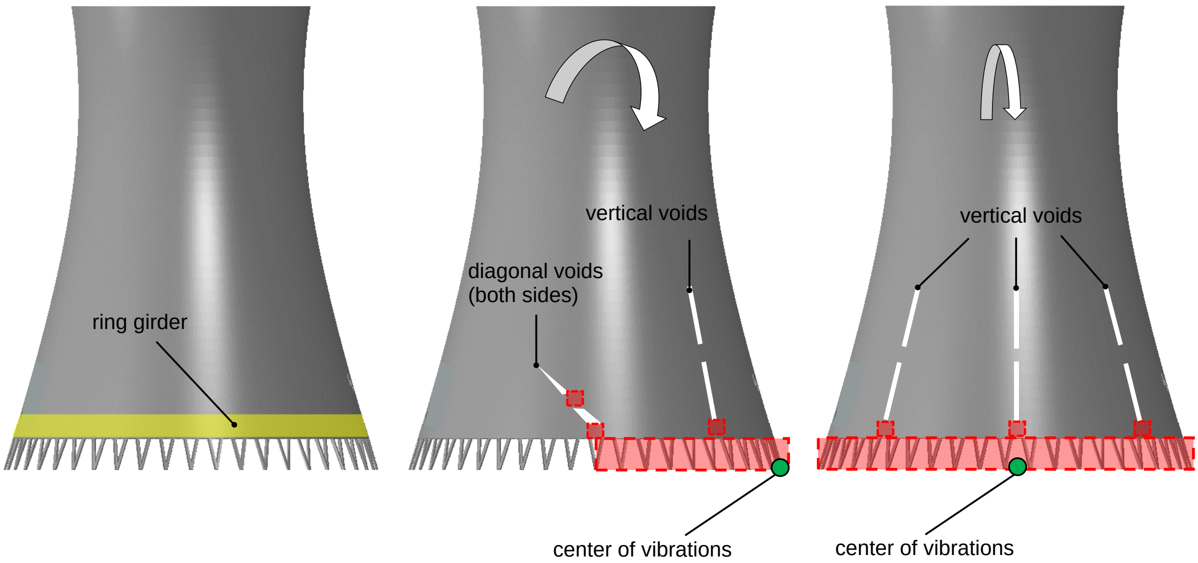

Figure 3: Philippsburg hyperbolic cooling towers: Demolishing concept

First, the cooling towers were pre-damaged (white cutouts) to ensure the desired direction of fall when blasting. At the lower edge of the cooling tower shell, the wall thickness was approx. 75 cm (ring girder - yellow). The girder was excluded from the pre-damaging. The explosive charges were placed in the areas marked in red and detonated as synchronously as possible. Half of the total of 96 V-pillars, five areas of the ring girder and the webs in the diagonal cutouts were blown up. The diagonal cutouts were intended to induce the tilting direction indicated in Figure 3. Through this so-called tilting-collapse movement, the cooling tower shell was supposed to hit the center of the vibration marked in green as precisely as possible. The impact should completely destroy the cooling tower shell, which was largely intact up to this point in time.

A rough calculation shows that the collapse of one cooling tower releases large amounts of energy.

$$E_{pot} = \mathrm{m} \cdot \mathrm{g} \cdot \mathrm{h} = 23617000\ \mathrm{kg} \cdot 9.81 \frac{\mathrm{m}}{\mathrm{s}^2} \cdot 65.3\ \mathrm{m} = 15129\ M\ (3.62\ t\ TNT) \qquad \textit{Equation 1}$$

The potential energy of a cooling tower (E_{pot}), which is released on impact, far outweighs the energy released by the explosive charges. On the one hand, this energy is consumed by the collapse process of the cooling tower shell, on the other hand it is dissipated as ground vibration. At the time of the blasting, the power plant is no longer in power operation, but fuel elements still present on the site must continue to be actively cooled. The functionality of the corresponding infrastructure on the power plant site must be ensured. The requirement to prove that the power plant is free of retroactive effects, especially with regard to ground-induced vibrations, was accompanied by the decision to demolish by blasting.

2. Avoiding retroactive effects – proof concept

SMP Ingenieure im Bauwesen GmbH (SMP) accompanied the demolition of the cooling towers of the Philippsburg nuclear power plant from an expert side. The task of SMP was, on the one hand, to conceptually contribute to the development of a proof concept for the non-retroactivity of the dismantling by blasting, on the other hand, in the structural examination of the corresponding verifications, which were provided by contractors (third parties) of the operator of the power plant.
The proof concept to exclude retroactive effects caused by ground vibrations had to consider the following aspects:

1. Avoiding danger to life
2. Safe containement of radioactive material
3. Ensuring the operation of fuel element cooling

The first mentioned point is about the danger of persons through (partial) collapse of buildings. The second point addresses the structural integrity of affected buildings. The third aspect, which was found to be largely decisive in the course of the investigations, considers the safe operation of relevant infrastructure for cooling active fuel elements stored on the power plant site. Buildings and plant components of relevant infrastructure were identified and classified by third parties according to their distance from the vibration centers (Figure 3 - green dots). The verification objective could be thus limited to the fulfilment of the requirement of a non-exceedance probability of a vibration level at a certain distance from the vibration centers. A location at a distance of 314 m to a vibration center proved to be decisive. This location is also referred to as point of interest (POI) in the following.
The design spectra for the load case earthquake (DEQ spectra) of the affected buildings were used as basis for the limit value of the tolerable vibration level. Basically, a verification concept on free field level was followed, which had two decisive advantages:

- Since all safety-relevant buildings are designed for the same earthquake, all buildings with a distance of more than 314 m to a vibration center are considered covered
- Accreditation of possible additional safety reserves, which can be justified by the usually favorable coupling factor at the transition from free field to foundation level

With regard to the load to be applied, it was decided to work with 95% quantile values. Additional safety factors were implicitly included on the resistance side by the requirement that the safety relevant buildings had not to exceed a proportionate level of the design earthquake. The parties involved agreed on a vibration level of 60 % DEQ in all directional components that should not be exceeded. The verification can thus be summarized as follows:

$$E_{c95,dir} \leq 0.6 \cdot DEQ_{dir} \qquad \text{Equation 2}$$

$E_{c95,dir}$: Characteristic value of the predicted induced ground vibration (95 % quantile) expressed as component spectrum

DEQ_{dir}: Design Earthquake (component) for Philippsburg power plant (according to (KTA 2201.4 2012)*)*

3. Prediction of the soil induced vibrations and vibration interference of the cooling towers

Basically two concepts for the preparation of a vibration prognosis were followed by the operator. Both concepts are based on an existing database of vibration velocity measurements at blast demolitions of hyperbolic cooling towers. The data set included a total of 10 cooling tower blastings and 165 velocity measurements. The data includes both free field and building measurement points. Most of the data were determined for tower blastings with significantly lower energy input (E_{pot}) in comparission to the Philippsburg towers. Evaluation points with mostly shorter distances to the vibration center than 314 m predominated.
The first method represents a forecast model purely on a statistical basis. Here, a forecast formula with two free parameters (E_{pot} and distance to the vibration center) was determined by the contractor. The procedure according to method 1 is explained exemplarily: First, a reference signal from the database is transformed into a spectral representation of the accelerations. The prognosis model requires as input variables the potential energy of a cooling tower of Philippsburg, as well as the distance to the center of vibration (here: 314 m). The prognosis model then provides a scaling factor. This factor is multiplied with the acceleration spectrum over the entire frequency range. This procedure is repeated for several suitable reference signals. The increase of the spectra to the 95 % quantile level is already taken into account. However, this procedure proved to be too conservative with respect to the prediction quality of the accelerations for larger frequencies. This is due to the fact that the existing data base is dominated

by vibration measurements with significantly shorter distances to the center of vibration and therefore an extrapolation of the signal to larger distances had to be performed. Due to the scalar modification of the reference signals over the entire frequency band, the frequency-dependent transmission behavior of the ground vibrations could not be mapped. The transmission behaviour of higher frequency signals was thus overestimated.
On the advice of SMP, the operator therefore decided to pursue an alternative method. This second method should include the transfer behavior of the ground vibrations as a frequency-dependent function (transfer function). The transfer function was determined numerically with the program Sassi. The input parameters were obtained from the existing soil expertise on the one hand and from shaker tests (Figure 4) on site on the other hand. The numerically determined transfer function for a signal transferred to the POI is shown in Figure 4. The procedure according to method 2 is explained exemplarily: First a reference signal from the database is transferred into a spectral representation of the accelerations. Then a scaling of the spectrum to take into account the existing potential energy of a cooling tower of Philippsburg with a factor over the whole frequency band is performed. Then a frequency-dependent adjustment of the reference signal is performed using the transfer function, which takes into account the distance of the reference signal from the POI. Furthermore, the procedure is analogous to method 1. Independent comparison calculations by SMP provided forecast spectra which differed less than 20 % from those of the operator. The acceleration spectra determined at the POI for the free field with method 2 were considered as the decisive basis for evaluation in the further course of the proof of the absence of retroactive effects.

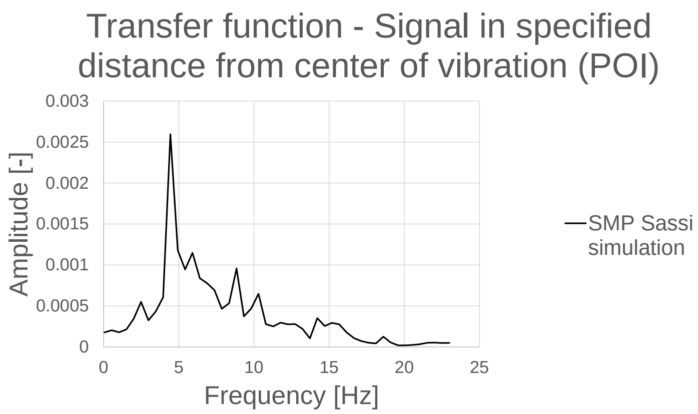

Figure 4: Left: Numerical determined transfer function; Right: Shaker tests on Philippsburg power plant site

Using method 2 it could be shown that the requirements according to Equation 2 could be met. However, this was only under the condition that there is no significant superimposition of the ground vibrations caused by the two collapse processes of the cooling towers. This circumstance is taken up again in section 4.

4. Simulation of the structural behavior of one cooling tower

Investigations to ensure a controlled collapse of one tower caused by blasting

Although there is already a lot of experience worldwide, but also in Germany alone, with the demolition of hyperbolic cooling towers, each cooling tower is a unique structure. In addition, the cooling towers in Philippsburg are the largest hyperbolic cooling towers to date, which were to be demolished in Germany by blasting. Above all, however, due to the required verification objective (Equation 2) it was therefore necessary to analyze the collapse process of the towers immediately after the blasting of the towers in advance in order to exclude an uncontrolled collapse process caused by the blasting as far as possible.

For example, an uncontrolled collapse could have caused a shift of the center of vibration to any point of the lower ring girder, which would have fundamentally changed the vibration prediction at the POI. Also the predicted debris corridor (Figure 1) could have shifted, which would have had a strong impact on safety aspects other than ground-induced vibration.

To exclude the possibility of an uncontrolled collapse of the towers due to the blasting, SMP carried out finite element simulations using the software Abaqus. An uncontrolled collapse was considered to be probably excluded if there is a significant tilting movement of the cooling tower shell without failure of the remaining columns. A numerical model of a cooling tower was created. The cooling tower shell was discretised by means of two-dimensional shell elements. The columns were discretised as beam elements. The support conditions were varied to adequately account for the not exactly quantifiable deformation behaviour of the soil around the column foundations. Within the framework of a best-estimate investigation, mean values of the material characteristics for concrete and reinforcing steel were used for the simulations. A post-hardening of the concrete was also taken into account. The reinforcement of the relatively low reinforced shell was considered in this model by a slight increase of the concrete tensile strength. A complete failure of the concrete from a compression of 3.5 ‰ was implemented. The material model for the columns was linear-elastic.

The modelling took into account the order in which the load was applied. First the dead weight was applied to the still undamaged tower. Then the pre-damaging was modeled, whereby load transfer effects occurred. Figure 5 shows the further course of the simulation.

The blast was modeled by abruptly removing the corresponding columns and ring girder sections (Figure 5 top left). A beginning crack formation process occurred in the area of the diagonal pre-damaging. The crack formation continued on both sides of the tower and a closed separation crack was formed over the entire circumference. The simulation was continued until the impact event of the separated tower shell occurred.

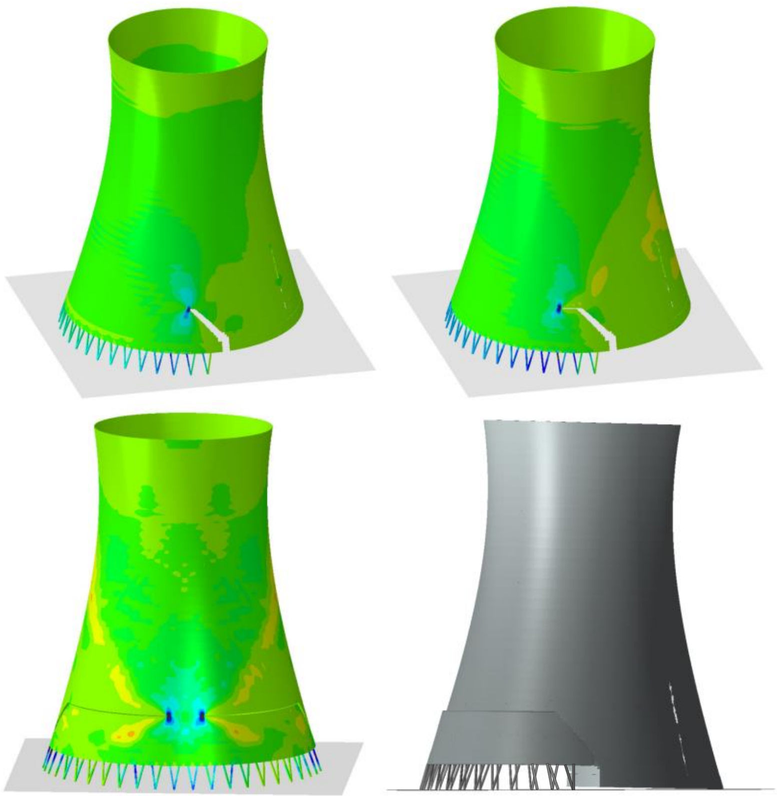

Figure 5: Simulation of dismantling by blasting (minimum in-plane stress – blue means high compression stress): Top left: Removing parts (blasting), cracking starts at diagonal pre-damaged area – Top right: Cracking continues; Down left: Both cracks meet and form one circumferential crack – Down right: Complete separation of the main rotating part of the shell and contact on the surface

The load bearing capacity especially of the columns below the diagonal pre-damaging (critical columns) was investigated up to the time of the formation of the closed circumferential crack (Figure 5 - down left). The internal forces were transferred into a moment normal force diagram and compared with the material resistance curve, which was simplified by a polygon course (Figure 6). The proof of the controlled collapse of a cooling tower under blast could be provided.

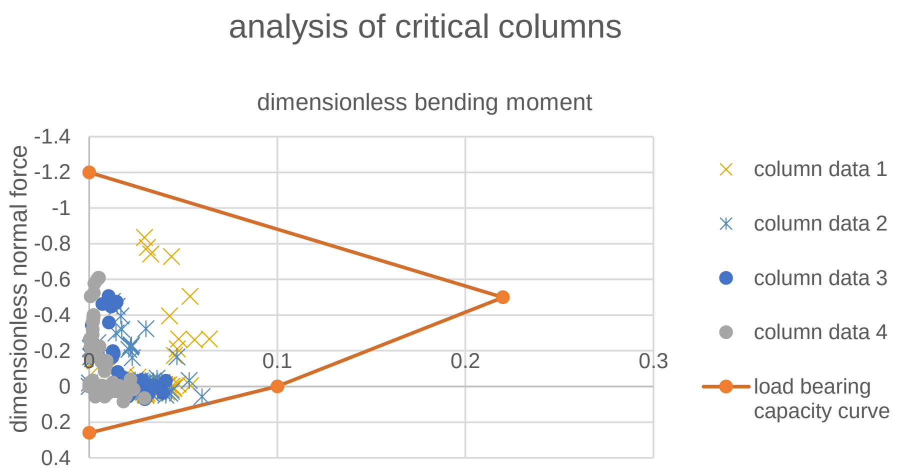

Figure 6: Structural analysis of critical columns during demolishing using section force and moment

<u>Investigations to exclude the collapse of second tower under soil vibrations induced by the first tower</u>

Investigations by SMP showed that the originally planned time staggering of only a few seconds between the blastings of the cooling towers could lead to considerable interference phenomena in the ground vibrations. Further investigations proved that only after a time offset of about 12 seconds of the blast releases a decoupling of the ground vibrations could be assumed. However, this now considerably longer time window made it necessary to exclude an uncontrolled collapse of the briefly remaining tower caused by the vibrations induced by the first tower.

For this purpose, SMP carried out finite element simulations using time history data on the cooling tower model in Abaqus. The model already presented was therefore reused and modified. Meridian and circumferential reinforcement was modelled using the rebar option for shell elements in Abaqus. Therefore an adaption of the concrete material behaviour concerning tensile strength was not neccessary any more. Mean values of the material characteristics for concrete and reinforcing steel were used.

Time history data was generated for all directional components on the basis of the results from section 3. The data was scaled with a safety factor calculated by third parties on basis of (DIN EN 1990 2010). On the safe side, the decisive distance chosen was the shortest distance from the vibration center of the first tower to the nearest column foundation of the second tower. This was approximately 100 m. Then spectrum-compatible time history acceleration data sets were generated which were applied on all column supporting points. The additionally applied wind load was significantly lower than the wind load from the as-built statics. In this way the fact was taken into account that the proof of non-collapse was only to be provided for the relatively short period of time lag between the two blasts. On the safe side, a duration of one day was assumed. The reduction of the applied wind load in comparision to the as-built statics was determined by third parties involved.

In the course of the investigations two aspects turned out to be decisive. On the one hand, the increasing crack formation in the concrete in the area of the pre-damaging due to the superposition of horizontal wind and vibration loads. On the other hand, the vertical load of the column foundations due to the superposition of dead weight and vertical vibration components.

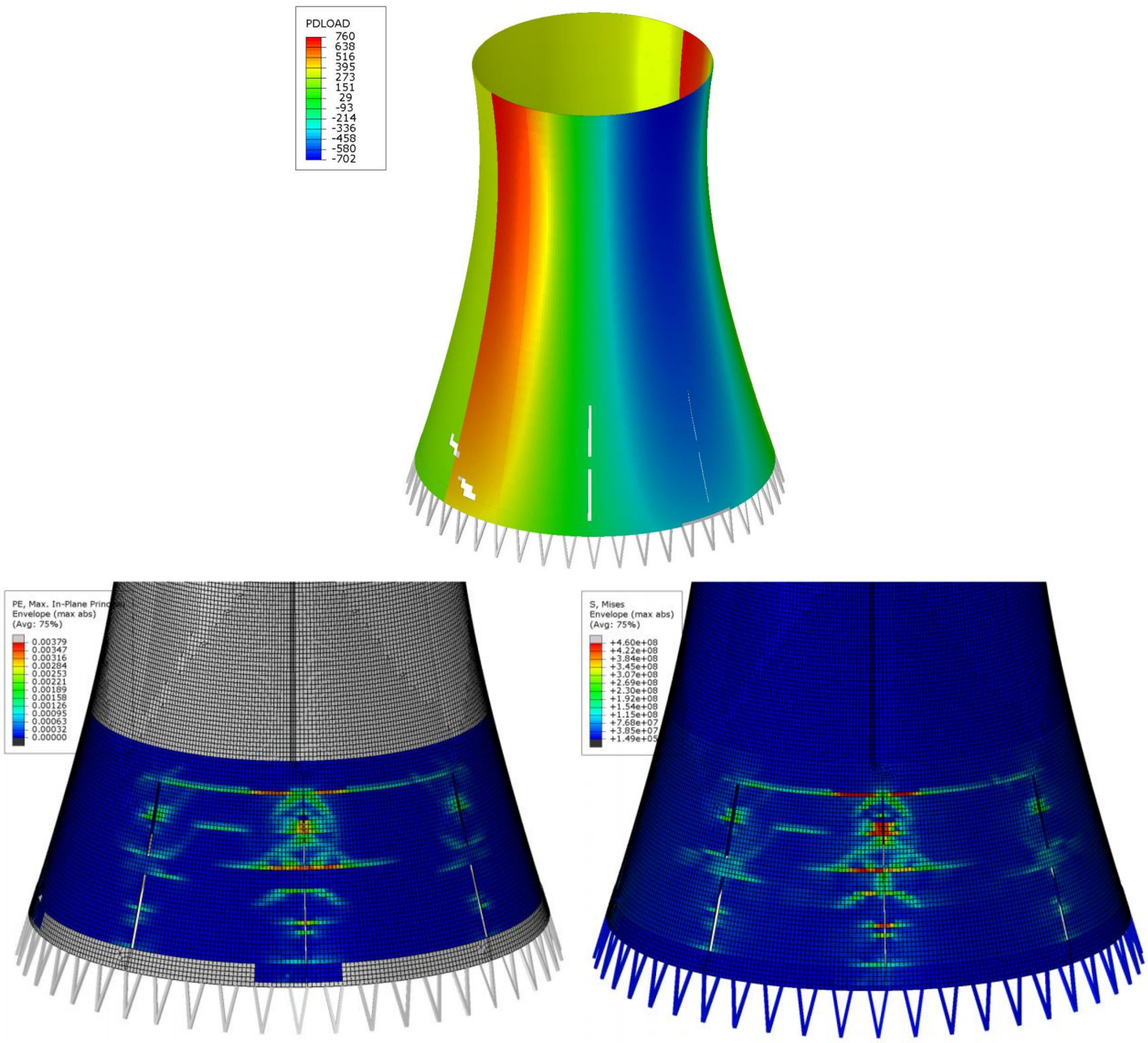

Figure 7: Simulation of tower under soil vibration and wind: Top left: Wind load [N/m²] negative value means pressure – Bottom left: Plastic strain in concrete as envelope; Bottom right: Maximum stress in reinforcing as envelope [N/m²]

Figure *7* shows one wind load scenario which was applied together with acceleration time history data at the supports. Plastic strain in the concrete at the area of the vertically pre-damaged part of the shell are shown in the bottom left of Figure *7*. The locations of plastic strain roughly correspond to the expected crack pattern (smeared crack modelling). Maximum tensile stress in the reinforcing is shown in Figure *7* bottom right. The locations where maximum stress occured correspond to the plastic strain pattern. As Yielding strength was set to 465 N/mm² it can be seen that no yielding occurred.
Figure 8 shows the extremal vertical reaction forces for all simulations being performed. The maximum reaction force (pressure) was around 14.8 MN which lead to a degree of utilization for the soil under the foundation of ~ 110 %. This was considered tolerable as all other foundations had much less loading at that specific time. The same reasoning also applies to the lifting forces that occur on the column with the least load.

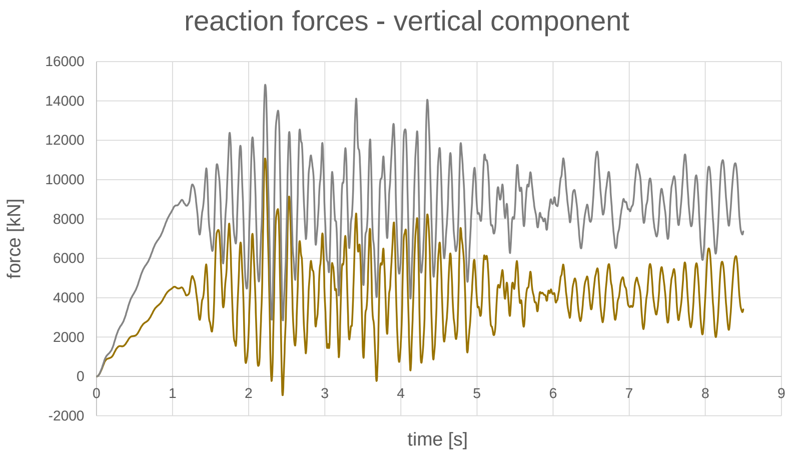

Figure 8: Extremal reaction forces (vertical component) – only critical column foundations are shown

The structural analysis of the columns was also performed0.
Based on the presented results with regard to the tower shell, the foundations and the columns, the proof of the non-collapse of the briefly remaining tower under combined wind and ground vibration load was considered to be provided.

5. Dismantling by blasting: Results

The blasting of the cooling towers in Philippsburg took place on the morning of May 14, 2020, with the ignition of the charges in each tower occurring practically synchronously. The time offset of the blastings of the two towers was about 12 seconds. The two towers failed exactly as calculated in advance, both with regard to the collapse process and the location of the vibration center. The remaining tower was not visibly affected by the ground vibrations induced by the impact of the first tower. In Figure 9 the crack formation process immediately after column blasting is shown, which corresponds very well with the results of the simulation (Figure 5).

Figure 9: Cracking in the tower shell immediately after the ignition of the explosive charges

The demolition of the blast was accompanied by third parties by means of measurement technology in order to check that the previously defined limit vibration level was maintained. An evaluation of the measurement results by SMP for selected measuring points could prove that the required verification target could be clearly met for all directional components. In addition, it was checked for relevant buildings whether the permissible vibration speeds according to (DIN 4150-3 2016) had been observed, which was also the case.

6. Conclusion

SMP Ingenieure im Bauwesen GmbH (SMP) accompanied the demolition of the two hyperbolic cooling towers of the Philippsburg nuclear power plant from an expert side. The task of SMP was, on the one hand, to conceptually contribute to the development of a proof concept for the non-retroactivity of the dismantling by blasting, on the other hand, in the structural examination of the corresponding verifications, which were provided by contractors (third parties) of the operator of the power plant.
This paper summarizes the verification process that ensures that there were no retroactive effects with regard to the vibrations caused by the collapse of the cooling towers. First, the definition of the desired level of security is outlined. The verification strategy is then examined. Particular attention is paid to the determination of reliable vibration predictions and numerical simulations of the structural behavior of the briefly remaining tower, which had to withstand the vibrations caused by the collapse of the first tower.
Vibration predictions performed by third parties used a method which combined empirical data and numerical simulations. The prediction proved that an exceeding of the desired level of security expressed as acceleration spectra was highly unlikely. The vibration predictions implicitly assumed, that the collapse of the towers happened in a very controlled manner and that no significant interference between the induced vibrations of the two towers occur. To validate these assumptions additional examinations were necessary.
These examinations were conducted by SMP using finite element models of one cooling tower. The models included material and geometrical non-linearities and were performed under dynamic loading. Concrete cracking and consideration of reinforcement as embedded elements was partly implemented too. The examination of the first numerical model demonstrated, that a controlled collapse of one cooling tower was very likely as long as the blasting charges detonated as synchronously as possible. To exclude interference effects of the soil vibrations a time offset of the two blasts had to be implemented. The second model showed that the briefly remaining tower could withstand the vibrations induced by the first tower and an uncontrolled collapse was unlikely.
The dismantling took place on May 14th 2020. Accompanying measurements showed that the level of induced vibrations were clear below the before defined tolerable level. The collapsing process could be accurately mapped by the numerical simulations.

Literature

DIN 4150-3. *Erschütterungen im Bauwesen - Teil 3: Einwirkungen auf bauliche Anlagen.* 2016.

DIN EN 1990. *Grundlagen der Tragwerksplanung.* 2010.

KTA 2201.4. *Auslegung von Kernkraftwerken gegen seismische Auslegungen.* 2012.