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Seismic Qualification of Class II Buildings of Nuclear Power Plants on the Basis of Non-Nuclear Seismic Codes

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1 ABSTRACT

The German nuclear code for the seismic design of class II buildings which may jeopardize class I buildings in case of failure allows the use of the national German seismic code for non-nuclear buildings. The nuclear code, however, refers to a version of the non-nuclear seismic code which is not valid any more. A revision of the nuclear seismic code is under way. Until the completion of this revision the user need some guidelines for the appropriate application of the presently valid national seismic code. This paper makes some proposals for the proper use of this code. The proposed methods are demonstrated at the revaluation of the main turbine building of a pressurized water reactor. The seismic strength of the building is checked and the seismic excitation at the location of equipment is calculated. The results are verified by use of a detailed analysis according the standard methods as required for class I buildings in nuclear industry.

2 INTRODUCTION

The design basis for the seismic qualification of class I buildings of nuclear power plants in Germany is the draft of KTA 2201.3 (1990). The required methods for the dynamic analysis of the buildings are the response spectrum modal analysis (RSMA) or time history methods. The design is based on force controlled approaches with linear response of the buildings. It is not allowed to take advantage of the inelastic energy absorption capacity of the building. The strength of the building and its parts is verified by use of the allowable stress concept with one global safety factor. For class II buildings (buildings not relevant for the safety of the nuclear power plant) which may jeopardize class I buildings and components in case of failure (nowadays called class IIa), the nuclear code KTA 2201.3 (1990) provides with the use of the national German seismic code DIN 4149 (1981) for seismic qualification of buildings other than nuclear buildings as it was legal at the publication of KTA 2201.3 (1990).

In the context of the harmonisation of the European technical codes, the national German codes were revised. The main focus of the revision is on the introduction of the load and resistance factor concept with partial safety factors and combination factors for actions and resistance. The German seismic code DIN 4149 (2005) as national application document of the European seismic code EC 8.1 (2003) was completely revised on this behalf with regard to the actual state of art.

At the time, the German nuclear codes are adapted to the revised building codes. The seismic nuclear code KTA 2201.3 (1990) needs a basic revision with respect to the actual state of art. Especially the application of the seismic code DIN 4149 (2005) for the qualification of class IIa buildings must be redefined. This paper gives supports for the application of the revised seismic code DIN 4149 (2005) in the context of the qualification of nuclear class IIa buildings. This is shown at the main turbine building of a pressurized water reactor. The main turbine building is classified as class II building. In this building, however, are highly energized vessels, the failure of which during earthquake may endanger class I buildings.

3 COMPARISON OF CODE VERSIONS

3.1 Design of buildings

The presently valid version of KTA 2201.3 (1990) allows the following procedure for the analysis of class II buildings which may affect class I buildings in case of failure:

- The proof of stability may follow the regulations in the German earthquake code DIN 4149 (1981), for non-nuclear structures.
- The building should be ranked as class 2 according DIN 4149 (1981).
- An earthquake of the site specific intensity should be used, differing from DIN 4149 (1981).

With the publication of the totally revised German earthquake code DIN 4149 (2005), the validity of the old version of DIN 4149 (1981) expired. The major alterations in the prevailing context are:

- DIN 4149 (2005) defines the use of two types of linear-elastic analysis, depending on the structural characteristic of the building: the (simplified) lateral force method and the (complete) modal response spectrum analysis. The comparable methods in DIN 4149 (1981) are the general method which defines equivalent static forces on the basis of the vibration modes, and an approximation procedure for the horizontal equivalent loads.
- In DIN 4149 (2005) the buildings are classified in 4 importance classes. Building class 2 according DIN 4149 (1981) is comparable to importance class III according DIN 4149 (2005). The importance factor for class III according DIN 4149 (2005) is $\gamma_I = 1.2$.
- DIN 4149 (2005) defines different earthquake spectra depending on the prevailing ground type. DIN 4149 (1981) uses one normalized response spectrum independent of the ground type. Instead the input horizontal acceleration is multiplied by a soil factor.
- DIN 4149 (2005) allows for the reduction of the earthquake load by a behaviour factor q depending on the ductility class and structural type. In DIN 4149 (1981) no such factor is explicitly defined. The shape of the spectrum, however, includes indirectly some ductility
- The structural response to the two horizontal excitations may be combined according DIN 4149 (2005) as square root of the sum of squares (SRSS rule) or by adding 30 % of the secondary response to the main response (30 % rule). In DIN 4149 (1981) are no superposition rules mentioned. Vertical excitations must be taken into account only for special cases.

A further major revision in the new codes is the use of the load and resistance factor concept with partial safety factors and combination factors instead of the use of the allowable stress concept in the old codes. The progress report KTA-GS-78 (2005) regulates the application of the load and resistance factor concept for nuclear power plants.

3.2 Design of equipment in buildings

The seismic design of mechanical and electro technical equipment is regulated in KTA 2201.4 (1990). The standard procedure for the calculation of the seismic excitation at the location of the equipment is the derivation of floor response spectra as input. This procedure is valid for class I equipment as well as class II equipment which may jeopardize class I equipment in case of failure. The stability and integrity (for pressurized components) must be guarantied.

The supporting structures and anchorage of such equipment may be designed according KTA 2201.4 (1990) for components or according KTA 2201.3 (1990) for buildings. It must be agreed for each individual case what code applies. In case of the application of KTA 2201.3 (1990) the reference to DIN 4149 (1981) holds also for the equipment. Both versions of the non-nuclear codes DIN 4149 (1981) and DIN 4149 (2005) refer to the design of non-structural elements. While DIN 4149 (1981) simply defines the seismic action by an acceleration of the non-structural element as 1.5-times the design ground acceleration, DIN 4149 (2005) presents a higher sophisticated formula, which incorporates the height of the non-structural element in the building and the basic periods of the non-structural element and the building.

4 SEISMIC QUALIFICATION OF TURBINE BUILDING

The seismic qualification on the basis of the German seismic code DIN 4149 (2005) is demonstrated at the main turbine building of a pressurized water reactor plant. The main turbine building is rated as class II building and therefore not designed for earthquake loads. In this building, however, are highly energized vessels like the feed water tank, the failure of which during earthquake may endanger class I buildings. In conjunction with the derivation of new site specific seismic spectra it was now required to re-evaluate the seismic strength of the building.

4.1 Description of the turbine building

The turbine building has the plane dimensions 88x48 m with the height of 33 m above ground. It has a shallow foundation with the base plate of 2 m thickness about 8 m below ground. It is a reinforced concrete structure, the stiffness of which is dominated by bending frames with additional walls around the staircases and in the basement below ground. Fig. 1 shows a cross-section of the building. The feed water tank is erected on level +12.0m in an annex beside the turbine hall.

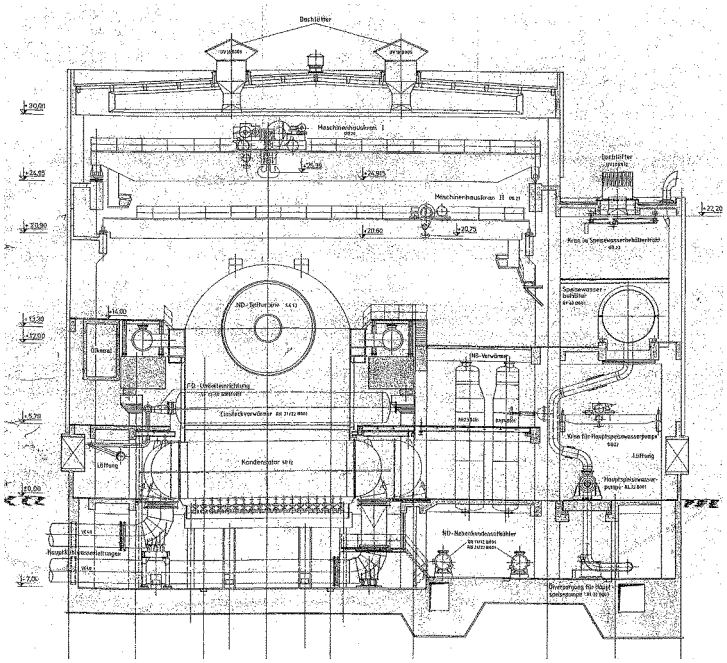

Figure 1. Cross section of turbine building

4.2 Earthquake excitation

In compliance with the requirements of KTA 2201.3 (1990) the site specific acceleration spectra are used, see Fig. 2. These spectra are valid at the base of the turbine building. They result from a deconvolution analysis. The horizontal spectrum describes the resultant excitation in any direction.

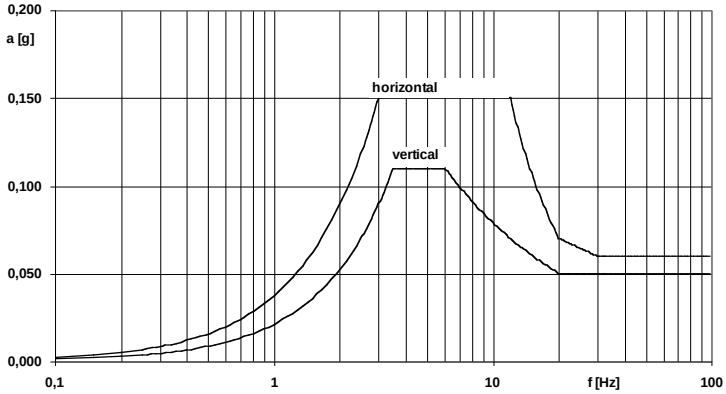

Figure 2. Site specific excitation spectra for D = 0.05 at level -9.0m

4.3 Design according DIN 4149 (2005)

For a quick review, the seismic qualification of the turbine building was at first accomplished by use of the actual German seismic code DIN 4149 (2005) as follows:

- The simplified (quasi static) lateral force method is used.
- Soil-structure interaction is omitted.
- Only horizontal excitations are regarded.
- The 5% spectrum is reduced to 7% for reinforced concrete.
- A behaviour factor of q = 1.5 for low ductility is used.

DIN 4149 (2005) allows to use plane models for buildings which are regular in plan. Although the condition for regularity in plan is only partly met, plane models are used here. Fig. 3 shows the models of one typical lateral frame and of the crucial longitudinal frame between turbine hall and feed water tract. The columns are clamped at their bases. The roof girder is hinged on the columns. The proportionate masses are lumped to the intersections of columns and beams.

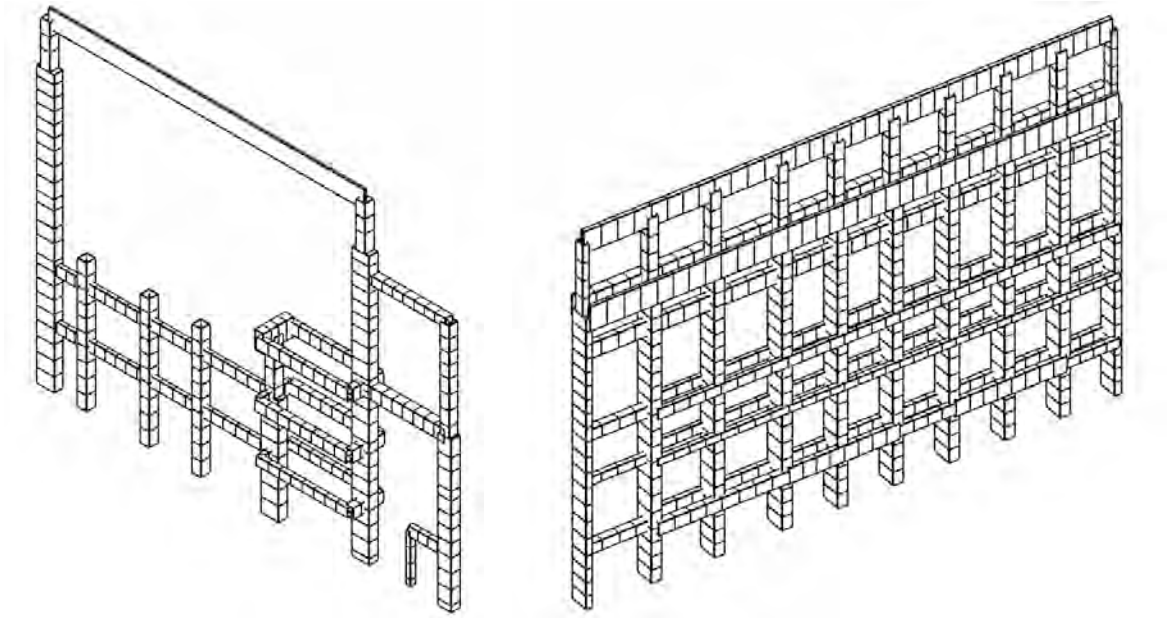

Figure 3. Plane models of selected lateral and longitudinal frames

The first modes of the two frames are shown in Fig. 5 and 6 of section 4.5. The basic frequencies are 0.83 Hz for the lateral frame and 1.01 Hz for the longitudinal frame. Because the stiffness of the overall building may be larger than comprised with the single frames (stiffening by the staircases and additional walls) and the basic frequencies are in the ascending branch of the spectrum, the calculated frequencies are enlarged by an estimated factor of 1.5. The belonging spectral accelerations are 0.037g for the lateral frame and 0.05g for the longitudinal frame. These accelerations are again enlarged by a factor of 1.5 to regard for the participation of higher modes. The design accelerations are:

$$S_d(f_1) = e \cdot a(f_1) \cdot \gamma_I \cdot S \cdot \eta / q \qquad \begin{array}{l} = 0.034g \text{ for the lateral frame} \\ = 0.046g \text{ for the longitudinal frame} \end{array} \qquad (1)$$

where
- $e = 1.5$ enlargement factor for the contribution of higher modes
- $\gamma_I = 1.0$ importance factor according KTA-GS-78 (2005)
- $S = 1.0$ soil factor for use of site specific spectra
- $\eta = [10/(5+\xi)]^{1/2} = 0.91$ for damping ratio $\xi = 7$ %
- $q = 1.5$ for low ductility

The horizontal seismic forces are distributed according DIN 4149 (2005) along the height of the building proportional to the shape of the fundamental mode. The height dependent load factors can be interpreted as accelerations in g-units.

4.4 Design according KTA 2201.3 (1990)

In a subsequent step the results of the simplified analysis according DIN 4149 (2005) were checked by the detailed analysis as it is required according KTA 2201.3 (1990) for class I buildings. The focal points in this analysis are:

- The response spectrum modal analysis (RSMA) is used.
- Soil-structure interaction is regarded.
- Horizontal and vertical excitations are investigated.
- The 5% spectrum is not reduced.
- The inelastic energy absorption is not regarded (behaviour factor q = 1.0)).

A 3-dimensional model of the turbine building with combined beam and plate elements is used, see Fig. 4. The base plate is modelled as slab with constant thickness. The soil is regarded by vertical and horizontal spring elements. The masses of the structural elements are calculated on the basis of the dimensions of these elements. The masses of the non-structural elements as well as 50% of the variable loads are distributed along the beams and columns.

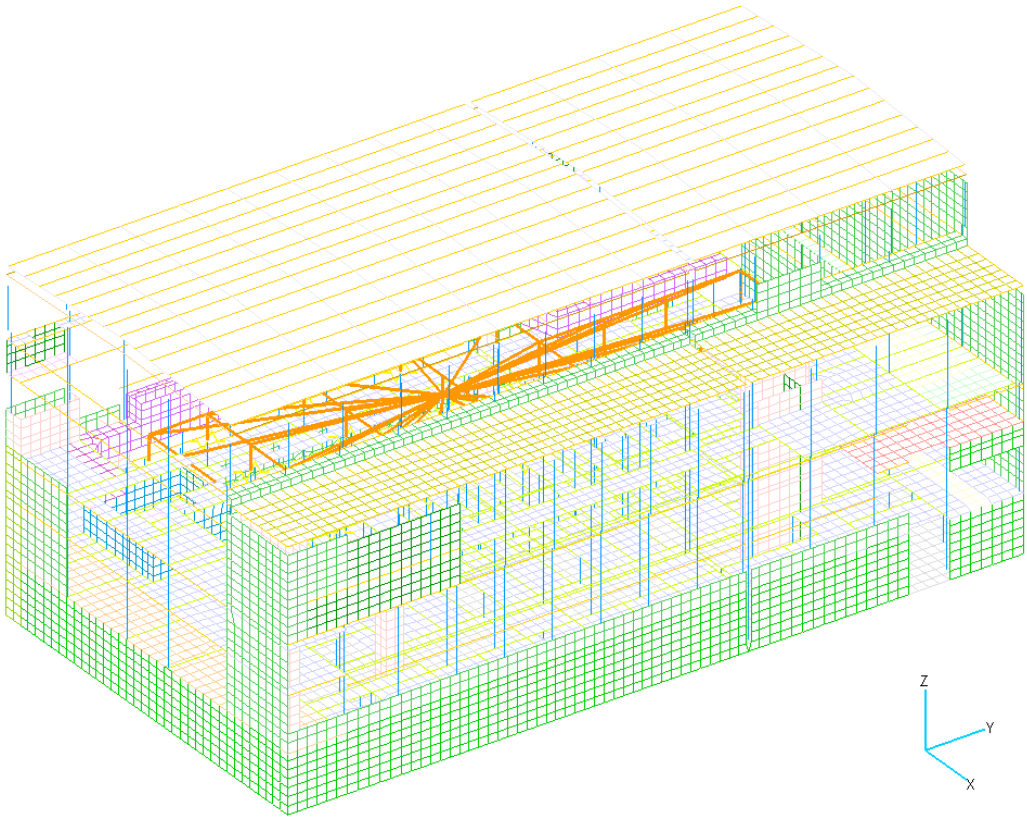

Figure 4. 3-dimensional finite element model of turbine building

4.5 Comparison of results

The fundamental modes are compared in Fig. 5 and 6 with the plane models at the left and the 3-dimensional model at the right. The basic coincidence of the shapes is good. As it was suspected, the frequencies are underestimated by the simplified plane models. It is shown that the selected enlargement factor of 1.5 of these frequencies comply very well with the results of the 3-dimensional model. The influence of the soil on the modes is negligible in this frequency range.

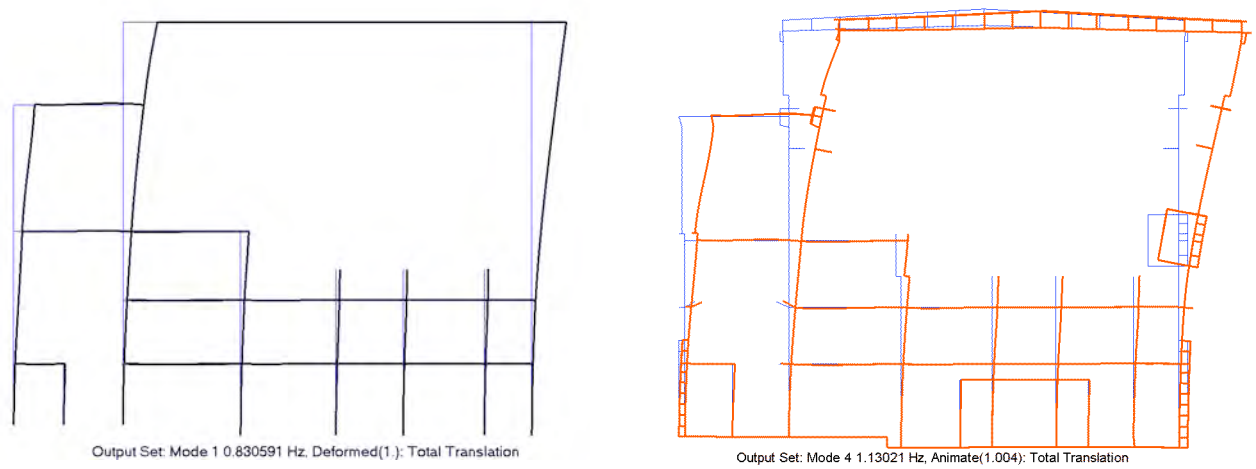

Figure 5. Fundamental mode for frame in lateral direction

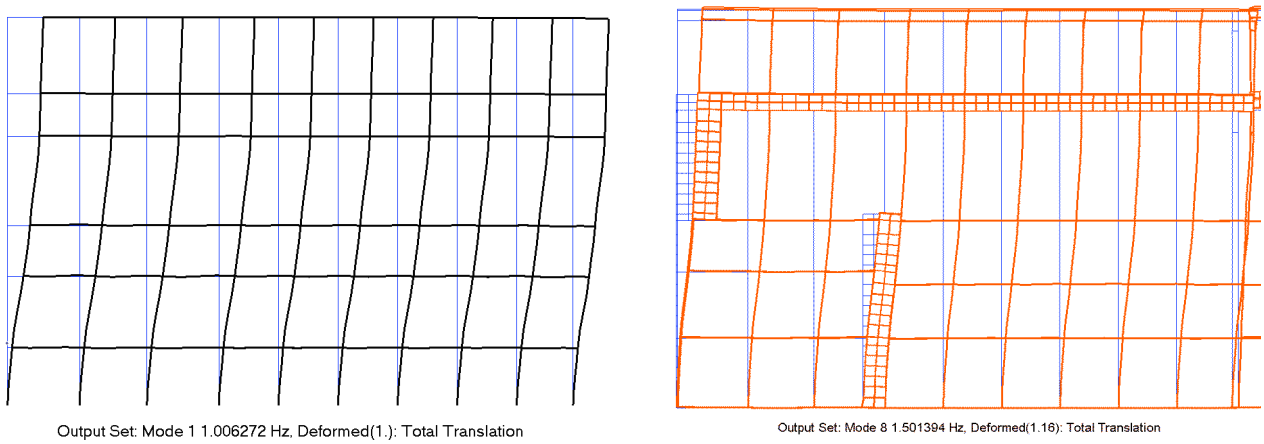

Figure 6. Fundamental mode for frame in longitudinal direction

Some selected results of the two analyses are compared in Table 1. These are the fundamental frequencies (with regard of the enlargement factor 1.5 for the plane model), the accelerations at the base and roof level of the frames and the section forces (bending moments and shear forces) at the base of the maximum loaded columns. The accelerations for the simplified analysis according DIN 4149 (2005) are derived from the distribution of the horizontal seismic loads along the height of the building. They are zero at the base of the frame because the omission of the soil-structure-interaction and maximum at the top of the frame. The comparison with the results of the detailed analysis according KTA 2201.3 (1990) for class I buildings shows, that the total accelerations are underestimated by the simplified analysis. The difference of the accelerations between top and bottom of the frames however is larger in the simplified analysis. This causes an overestimation of the bending moments at the base of the columns, while the shear forces are underestimated for the lateral frame.

Table 1. Comparison of selected results

		frequency	acceleration [m/s^2]		bending mom.	shear force
		[Hz]	base	roof	[kNm]	[kN]
lateral frame	DIN 4149 (2005)	1.5·0.83 = 1.2	0	1.44	2421	208
	KTA 2201.3 (1990)	1.13	0.75	1.75	1830	265
longitudinal frame	DIN 4149 (2005)	1.5·1.01 = 1.5	0	0.89	1866	582
	KTA 2201.3 (1990)	1.50	0.72	1.35	931	220

5 STABILITY OF FEEDWATER TANK

The feed water tank is erected on a reinforced concrete grid at level +12.0m. It is a horizontal cylindrical vessel with diameter 4.0 m and length 48.0 m on 5 saddle bearings. The saddle bearings are bedded on elastomere pads to prevent restrained forces during thermal expansions. In lateral direction the horizontal earthquake forces are transferred to the structure by contact between the saddles and the reinforced concrete beams at each saddle bearing. In axial direction, however, the earthquake forces are transmitted to the structure by thrust bearings at the middle saddle only.

To check the strength of the thrust bearings the axial loads on these bearings are needed. These loads are calculated by multiplication of the masses of the tank including impulsive part of the water and the mass of the convective part of the water (sloshing part) with the respective accelerations. The accelerations are calculated by use of the formula in DIN 4149 (2005) for non-structural elements:

$$S_a = a_g \cdot \gamma_I \cdot S \cdot \left[\frac{3\left(1 + z/H\right)}{1 + \left(1 - T_a/T_1\right)^2} \right] \qquad (2)$$

where
a_g	= 0.6 m/s^2	design ground acceleration
γ_I	= 1.0	importance factor according KTA-GS-78 (2005)
S	= 1.0	soil factor for use of site specific spectra
z	= 19 m	height of the attachment of the equipment
H	= 38 m	height of the building
T_a		fundamental period of the equipment
T_1		fundamental period of the building

(Remark: In the formula in DIN 4149 (2005) the amplification factor in the squared brackets is reduced by 0.5. Because the validity of this reduction is under discussion it is omitted here. For equipment with a high fundamental period respectively low frequency, as it is the case for the convective part of the water, the amplification factor is be negative, if the reduction by 0.5 is regarded)

As long as the fundamental periods are not known, an upper limit for the acceleration S_a is achieved with the assumption of resonance between equipment and building, that is for $T_a = T_1$. This results with the above shown parameters to S_a = 2.7 m/s^2. In the present case the modal parameters of the feed water tank are known from a detailed finite element analysis of the tank on the saddle bearings. The fundamental period for axial vibration is T_a = 0.25 s (f_a = 4.0 Hz). The fundamental period for water sloshing according EC 8, Part 4 (2007) is T_a = 7.7 s (0.13 Hz). With T_1 = 0.67 s (1.5 Hz) for the fundamental period of the turbine building for vibrations in longitudinal direction the accelerations according eqn (2) are

S_a = 1.94 m/s^2 for the tank with impulsive part of the water

S_a = 0.03 m/s^2 for the convective part of the water

With the masses m_T = 244 t for the empty tank including saddle bearings, m_i = 76 t for the impulsive part and m_{c1} = 274 t for the convective part of the water, the total axial force on the thrust bearing by linear superposition of the impulsive and convective parts is:

$$F_a = 1.94 \cdot 244 + 1.94 \cdot 76 + 0.03 \cdot 274 = 629 \text{ kN} \qquad (3)$$

To check these results the 3-dimensional model of the building is supplemented by the tank as elastic beam on the saddle bearings. The saddle bearings are modelled by spring elements between tank and building. The stiffness of these springs are adapted to achieve the fundamental periods of the tank on the saddle bearings as they were calculated by the detailed finite element model of the feed water tank. The results of this coupled analysis are:

S_a = 1.88 m/s^2 for the tank with impulsive part of the water

S_a = 0.03 m/s^2 for the convective part of the water

F_a = 569 kN

The agreement of the accelerations between the simplified analysis according DIN 4149 (2005) and the analysis with the coupled 3-dimensional model is very good. The axial force on the thrust bearing is lower in the coupled analysis because the participation of the elastomere pads below the saddle bearings.

6 CONCLUSION

The following approach for the application of the actual German seismic code DIN 4149 (2005) is suggested:

1. **Seismic action:** The site specific response spectra should be used instead of the spectra as defined in DIN 4149 (2005). This complies with the requirement of KTA 2201.3 (1990) to increase the security to a level that is valid for the complete plant. It is state of art in nuclear industry to use an earthquake with a reference probability of $1 \cdot 10^{-5}$/a as safe shutdown earthquake, while the design earthquake according DIN 4149 (2005) is defined for a reference probability of $2 \cdot 10^{-3}$/a.
2. **Components of seismic action:** If the horizontal response spectra are defined for the components of the seismic action, the structural response may be combined according DIN 4149 (2005) by the SRSS rule or alternatively by the 30 % rule. If a resultant horizontal response spectrum is defined, as in the preceding example, the respective maximum response must be evaluated without superposition of the two responses. The vertical component of the seismic action should be taken into account only according the criteria in DIN 4149 (2005).
3. **Analysis method:** Both methods, the simplified lateral force method as well as the complete response spectrum modal analysis should be applicable according the criteria in DIN 4149 (2005). In case of re-evaluation of existing structures as in the preceding example, the use of the lateral force method is preferred. However the design acceleration may be enlarged to account for the contribution of higher modes.
4. **Importance factor:** The importance factor is defined in KTA-GS-78 (2005) to $\gamma_I = 1.0$. This is justified because the enhanced importance is already regarded by the increased earthquake level.
5. **Behaviour factor:** The behaviour factors as defined in DIN 4149 (2005) should be applicable. At least a minimum factor of $q = 1.5$ for ordinary reinforced concrete and steel structures should be allowed in all cases. An upper limitation of the behaviour factor may be discussed.
6. **Design of equipment:** The effect of the seismic action on equipment in buildings may be determined by applying the formula of DIN 4149 (2005) for non-structural elements. The behaviour factor of the equipment, however, should be limited to $q_a = 1.0$. The soil factor is $S = 1.0$ for use of the site specific response spectrum.

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