Critical Filling Levels of Silos and Bunkers in Seismic Design

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SUMMARY:
In seismic design of silos and bunkers a filling level at maximum is assumed to be governing. Due to the big stored masses of the bulk solid the lowest natural frequency of the containing structure is very small. When using response spectra method given in Eurocode 8 for design, the vibration periods are larger than the parameters T_C or even T_D, which describe the shape of the acceleration function. This results in comparatively small accelerations and subsequently seems to result in a comparatively small base shear. Therefore, if filling levels are considered that are well below maximum, the lowest natural frequency of the structure will increase, thus leading to higher accelerations to be applied.
We report on a study where the question of a critical filling level is investigated. Practical examples from a recently built coal-fired power plant are given.

Keywords: silos, bunkers, EC8, response spectra, coal-fired power plant, foundation loads

1. INTRODUCTION

Power plants are often built within earthquake areas. Therefore, the buildings are designed to also withstand horizontal and vertical loads triggered by earthquakes. In coal-fired plants the bunker house which is housing the coal above the coal mills is loaded by the dead weight of the bunkers and the individual coal filling level. These loads are governing and trigger the highest horizontal loads during an earthquake.

To calculate the horizontal earthquake loads of such structures the maximum possible weight of the structure combined with the full bunker filling is combined with the lowest natural frequency and hence with the corresponding horizontal acceleration as stated within many design guidelines, e.g. EC8. Sometimes the complicated structure along with a complex mass distribution makes it difficult to determine the mass corresponding acceleration, according to the response spectra method. There are many regulations in EC8 which are difficult to meet in plant design as mentioned in Knoedel/ Hrabowski (2011).
This often leads to multiplying the mass with the maximum possible acceleration and to the highest forces not only on each individual member but also on the foundation to be on the safe side. By using a more sophisticated method where the different bunker filling levels are taken into account, these forces can be reduced and therefore, foundation sizes and costs will be minimised.

This paper reports on reducing the foundation load of a bunker house using modal analysis and by taking the bunker filling level into account. The resulting foundation loads will be compared to the results of the response spectra method using the mass corresponding horizontal acceleration on one hand and the maximum acceleration on the other hand.

2. BACKGROUND AND PROJECT DESCRIPTION

The task which was given during the design process of a coal fired power plant was to optimise the steel structure for coal bunkers with a nominal capacity of 1730 metric tons and reduce the foundation loads where possible. One of many design improvements was the more sophisticated load assumption within the earthquake load case. Until then a common load assumption was taken as illustrated below in Eqn. 2.1, which corresponds to EC8 Eqn. 4.11.

$$F_i = F_b * z_i * m_i / \Sigma(z_j * m_j) \qquad (2.1)$$

The overall horizontal force F_b (base shear) is calculated by multiplying the overall seismologic mass of the structure with a function where the Period T_i of the eigenfrequency of the structure is taken into account as shown in Eqn. 2.2, which corresponds to EC8 Eqn. 4.5.

$$F_b = S_d\,(T_i) * M * \lambda \qquad (2.2)$$

Therefore, the horizontal force for the earthquake load case is governed by the overall mass and the eigenfrequency of the structure which is taken into account with the calculation in Eqn. 2.3 and follows the graph in Fig. 1, where $T_C < T < T_D$ according to EC8 Eqn. 3.15:

$$S_d = a_g * \gamma_i * S * \beta_0 * T_C/(q*T) \qquad (2.3)$$

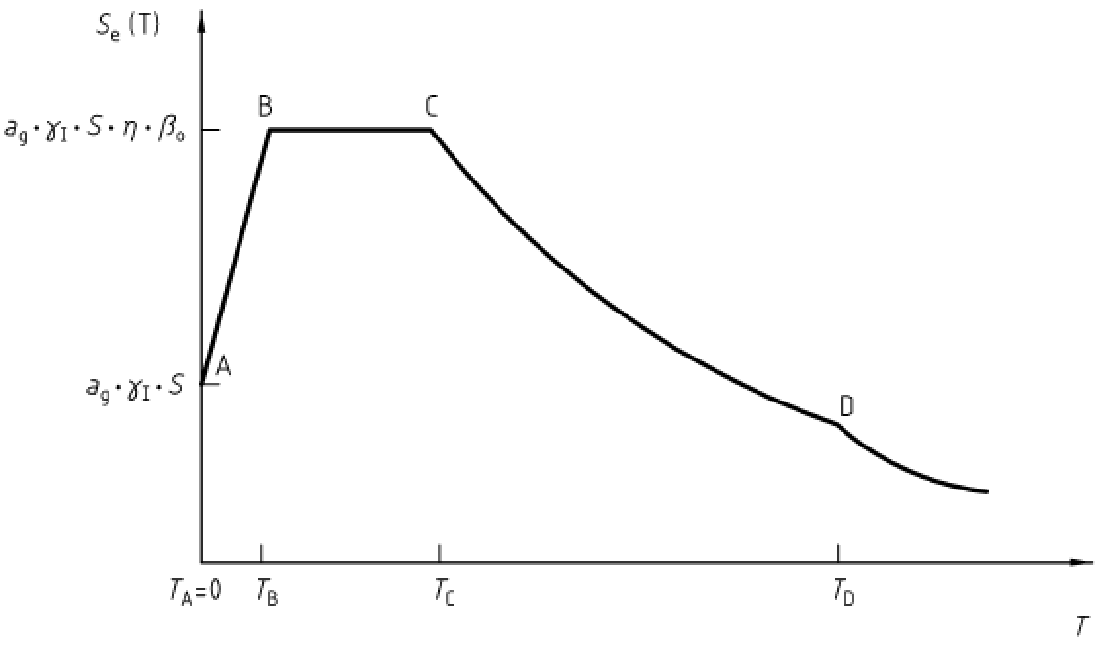

Figure 1. Elastic response spectrum according DIN 4149 Fig. 4/ comp. EC8 Fig. 3.1

For the comparison of the results, Eqn. 2.4 and Eqn. 2.5 are used.

When $T < T_B$ (corresponding horizontal acceleration):

$$S_d = a_g * \gamma_i * S * (2/3 + T/T_B * (2.5/q - 2/3)) \qquad (2.4)$$

When $T_B < T < T_C$ (maximum horizontal acceleration) according to EC8 Eqn. 3.14:

$$S_d = a_g * \gamma_i * S * (2.5/q) \qquad (2.5)$$

While it became apparent, that different filling levels within the coal bunker create different vertical loads, the horizontal loads will need to be adjusted accordingly. Using the modal analysis provided by a software program several filling levels were calculated between the two extremes of empty and full bunkers. The different filling levels used in this example were represented by the factors 1.0, 0.9, 0.75, 0.5, 0.25 and 0.1 of the maximum bunker filling. The changes in height of the centre of gravity were neglected on the safe side. The frequency of every case was extracted and the horizontal load calculated from that. Therefore, the different filling levels generated different eigenfrequencies of the structure which resulted in different acceleration of the different masses. For the overall horizontal load the maximum load of all different filling levels was taken.

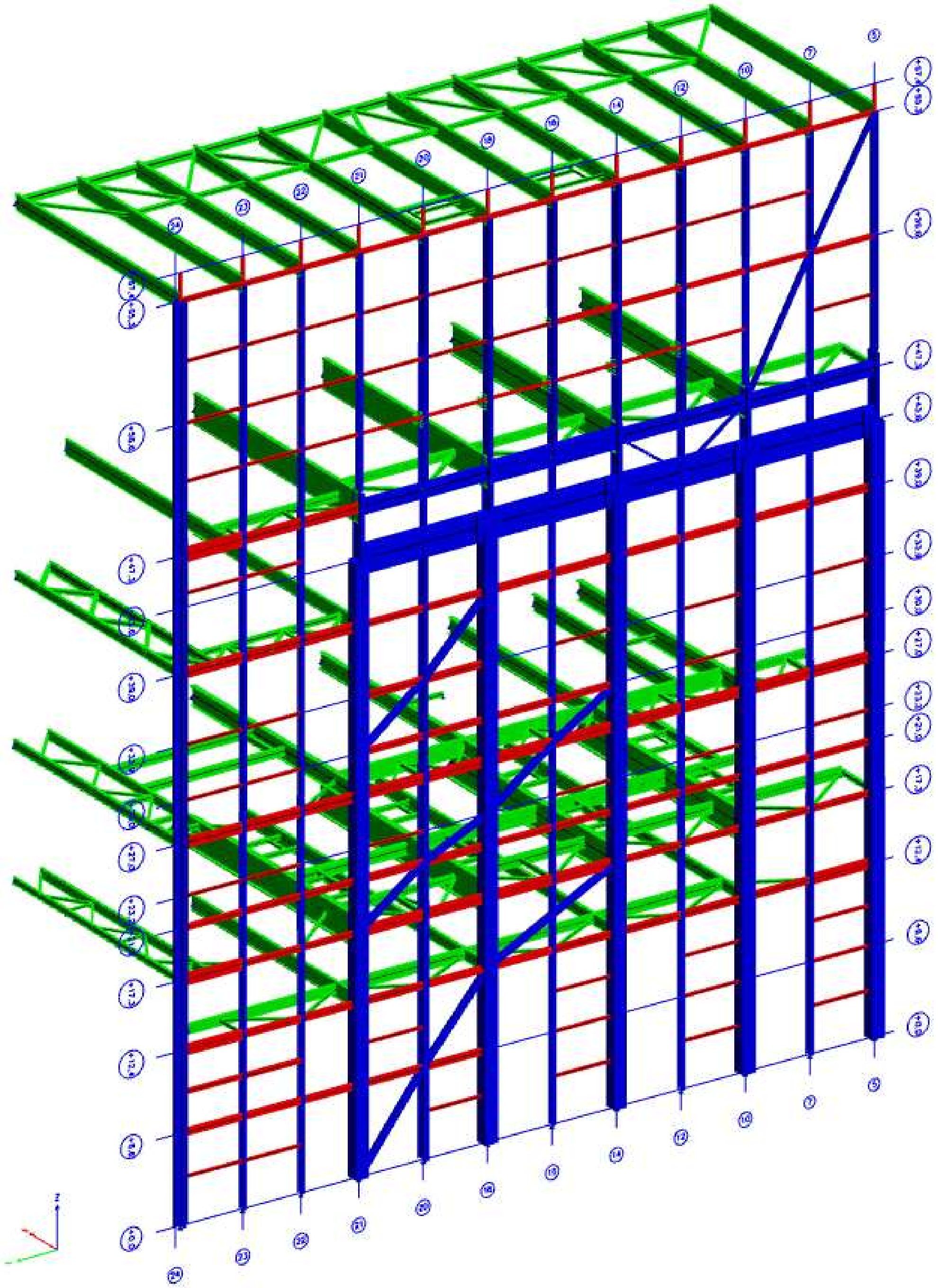

Figure 2. Structural model for a typical wall structure of the bunker house

Fig. 2 illustrates a wall of the bunker house with the horizontal bracing as modelled in the structural design program. To illustrate the differences between the two methods a simplified model is used as shown in Fig. 3.

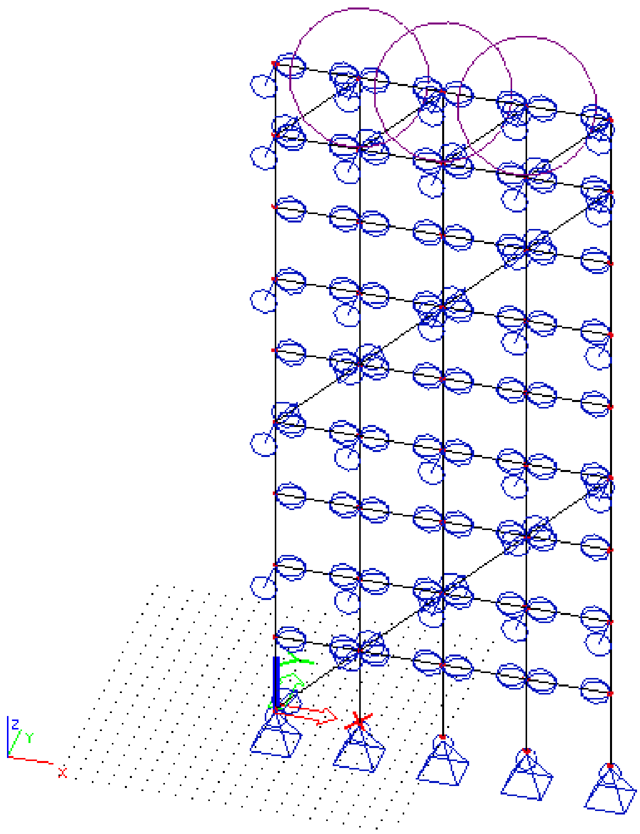

Figure 3. Simplified structural model for showing the differences in earthquake design

All the calculations are based on the same parameters as given in Eqn. 2.6 to Eqn. 2.14. This is according to the German earthquake code (DIN 4149) which also conforms to EC8 in the version of 2006, not with the version of 2010.

$$S = 0.75 \quad (2.6)$$
$$a_g = 0.4 \text{ m/s}^2 \quad (2.7)$$
$$\beta_0 = 2.5 \quad (2.8)$$
$$q = 1{,}5 \quad (2.9)$$
$$T_B = 0.1 \quad (2.10)$$
$$T_C = 0.5 \quad (2.11)$$
$$T_D = 2.0 \quad (2.12)$$
$$\eta = 1.0 \quad (2.13)$$
$$\gamma_1 = 1.0 \text{ (for simplified model, 1.2 as used in actual design)} \quad (2.14)$$

The simplified model shows a steel framework wall which is 20 m long and 45 m high. The columns have quadratic hollow cross sections 1000x1000x40x40 mm and a clearance of 5 m. The beams between the columns have I- shaped cross sections and are 800 mm high, 300mm wide, with a flange thickness of 30 mm and a web thickness of 15 mm. They have a clearance of 5 m as well. The diagonal members have a quadratic hollow cross section of 400x400x15x15 mm. This leads to a moment of inertia for the overall cantilever structure of 34.56 m^4 and with Young's modulus of 21000 kN/m^2 for steel to a stiffness of 7257600 MNm^2. All members are modeled with hinged ends including the connections to the basement well knowing that in reality there is a neglected stiffness reserve by elastic restraint at the bottom of each column as discussed in Knoedel/Mueller, et al. (2011).

In another recent paper it is shown, that EC8 is lacking regulations for higher strength steels. These seem to have a smaller behaviour factor by having 'overstrength' compared to lower grade steels as shown in Knoedel/Hrabowski (2012).
The weight of the bunkers and their filling add up to 17330 kN. This weight is modeled on 3 knots at the height of 45 m. Within the simplified model these are the only vertical loads, respective masses which are considered and therefore will be accelerated horizontally by an earthquake.

3. CALCULATION USING RESPONSE SPECTRA METHOD

The eigenfrequency of a cantilever wall with a horizontal load on its top can be calculated with the following Eqn. 3.1, as mentioned in Knoedel (2011):

$$f = \pi/2*(3*EI/(M*L^3))^{0.5} \tag{3.1}$$

With this equation and the above mentioned stiffness for the simplified model we get the following eigenfrequency f as given in Eqn. 3.2, and the period T as given in Eqn. 3.3.

$$f = 18.4 \text{ /s} \tag{3.2}$$

$$T = 0.054 \text{ s} < T_B \tag{3.3}$$

For $T < T_B$, the horizontal acceleration has to be calculated by equation 2.4 and results in Eqn. 3.4:

$$S_d\,(T) = a_g * \gamma_i * s * (2/3 + T/T_B * (2.5/q - 2/3)) = 0.36 \tag{3.4}$$

With this acceleration and the mass of 1733 to, the horizontal earthquake load on the top of the wall, we get, is H = 624 kN and therefore a maximum vertical tension force on the foundation of V = 1375 kN as illustrated in Fig. 4.

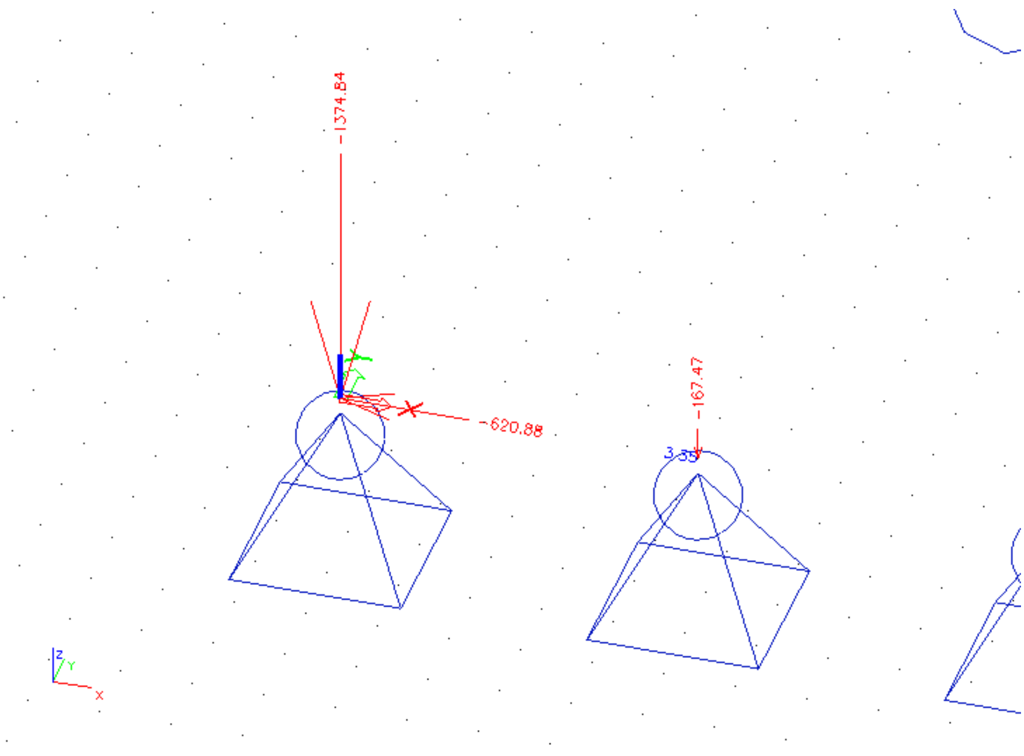

Figure 4. Horizontal and vertical foundation forces calculated with $S_d(T) = 0.36$

The maximum acceleration for a period $T_B < T < T_C$ is calculated with equation 2.5:

$$S_d = 0.5 \text{ m/s}^2 \tag{3.5}$$

Considering this acceleration and the mass of 1733 to, the calculated horizontal force on the top of the wall is H = 867 kN and therefore a maximum vertical tension force on the foundation of V = 1967 kN as shown in Fig. 5.

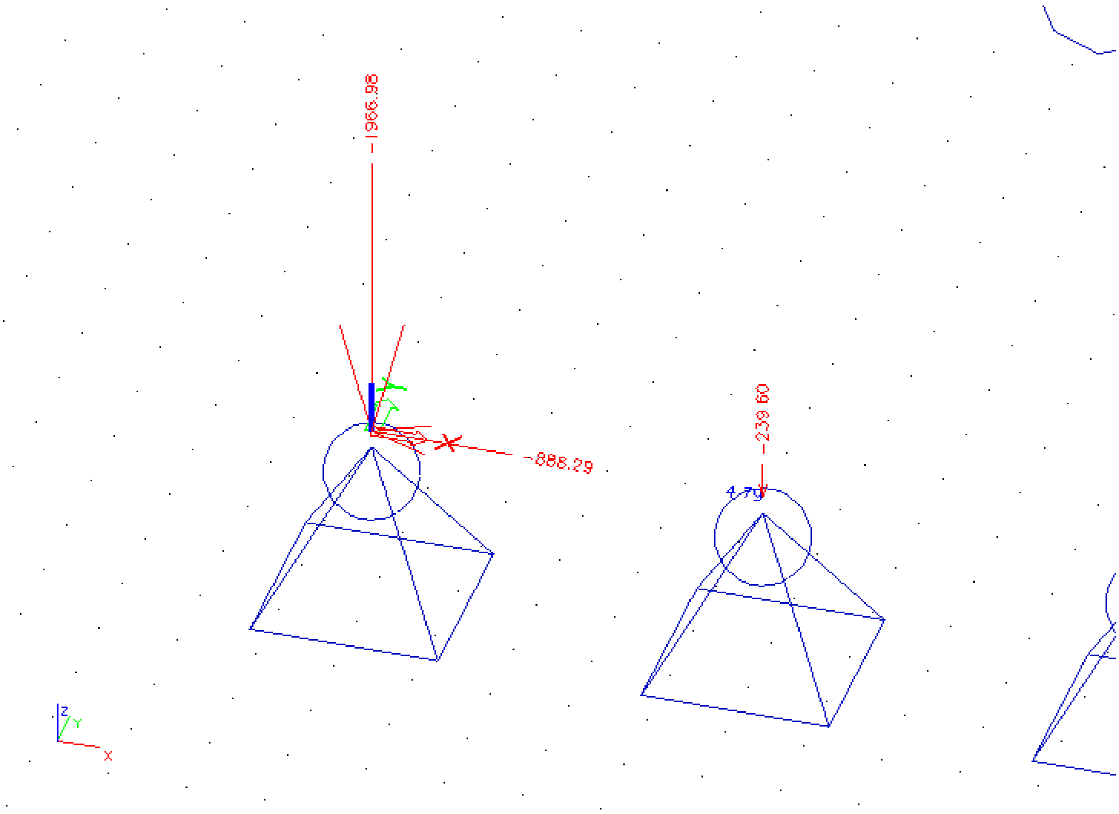

Figure 5. Horizontal and vertical foundation forces calculated with $S_d(T) = 0.5$ m/s^2

The difference between the mass corresponding acceleration and the maximum acceleration leads to a remarkable change in the foundation loads and the efficiency of the structure.

4. CALCULATION USING MODAL ANALYSIS

The modal analysis considers that every load in one system having a different eigenfrequency and is accelerated to a different direction at one time. To carry out the modal analysis in this case we used the method which is taking into account the answer of the eigenmode (R_j) combined with the maximum answer of all eigenmodes ($R_{jmax)}$:

$$R_{tot} = (R^2_{(jmax)} + R^2_{(j)})^{0.5} \tag{4.1}$$

For this method being a conservative calculation for our example, it is considering the eigenmode together with the maximum accelerated load, which we figured out with some case studies in comparison to SRSS- and CQC- method. Despite that we calculated smaller foundation loads than using the response spectra method as can be seen in Fig. 8.

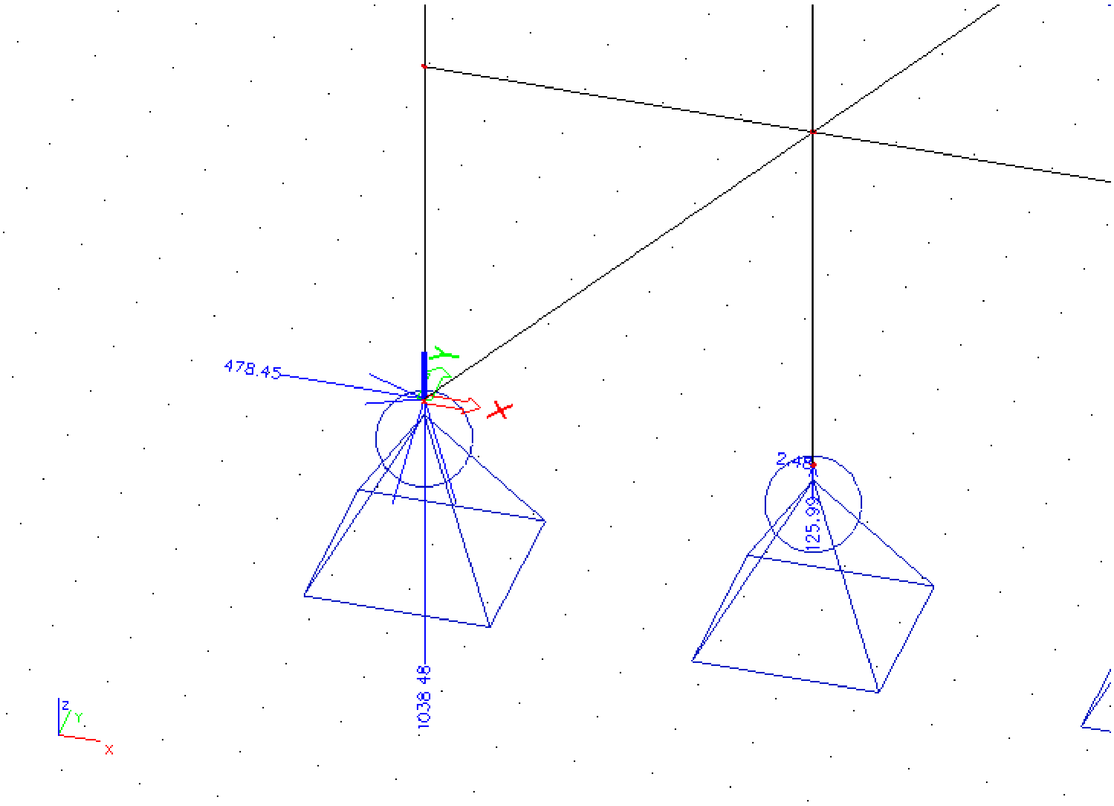

Figure 6. Horizontal and vertical foundation forces calculated with modal analysis and 100% filling

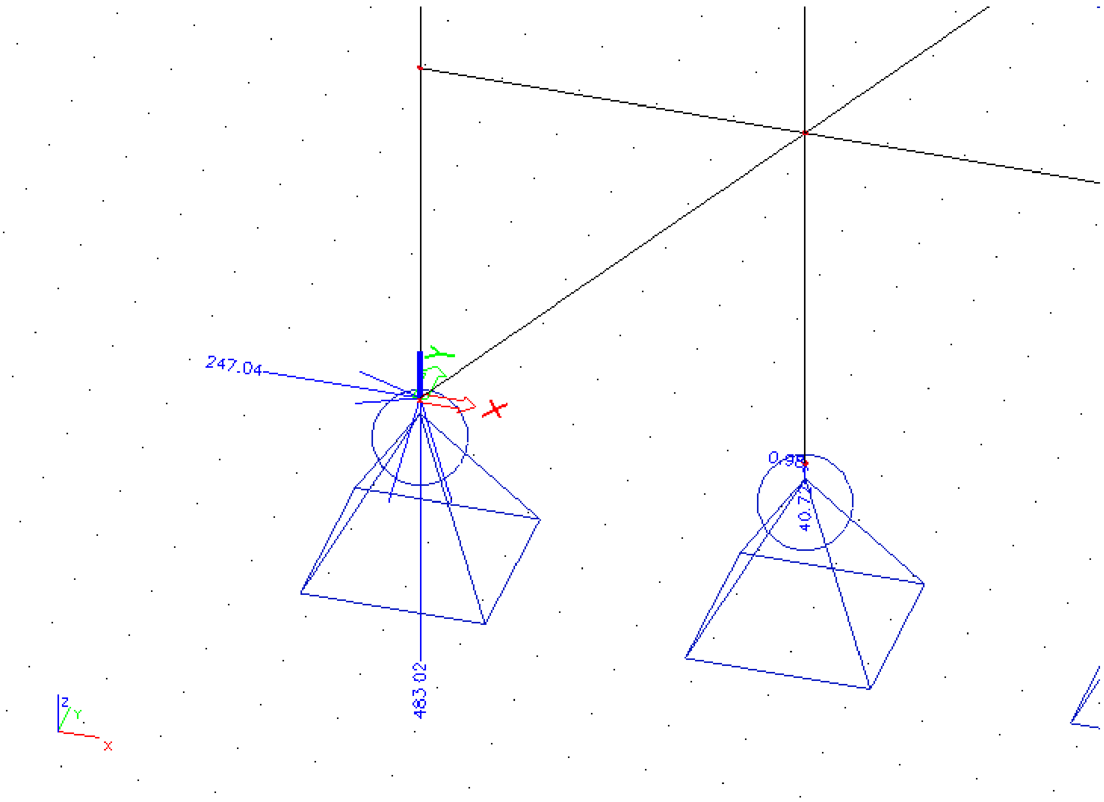

Figure 7. Horizontal and vertical foundation forces calculated with modal analysis and 10% filling

5. RESULTS

We can point out, that due to our investigations the maximum mass, not leading to the maximum acceleration, results in the maximum horizontal loads and the maximum foundation forces for simple structures and mass distributions as cantilever walls with mass on the top (SDOF) regarding all model uncertainties, which are needed to employ response spectra method with a SDOF single degree of freedom oscillator mentioned in Knoedel/Heß (2011).

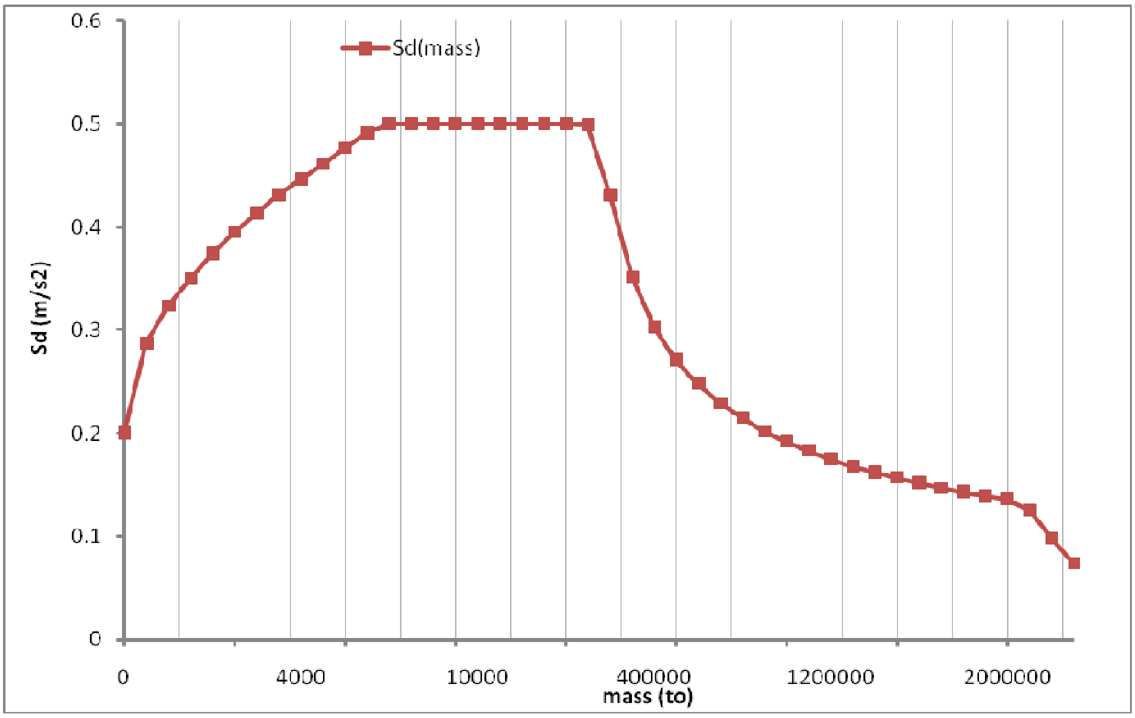

Figure 8. Example for Sd depending on the mass

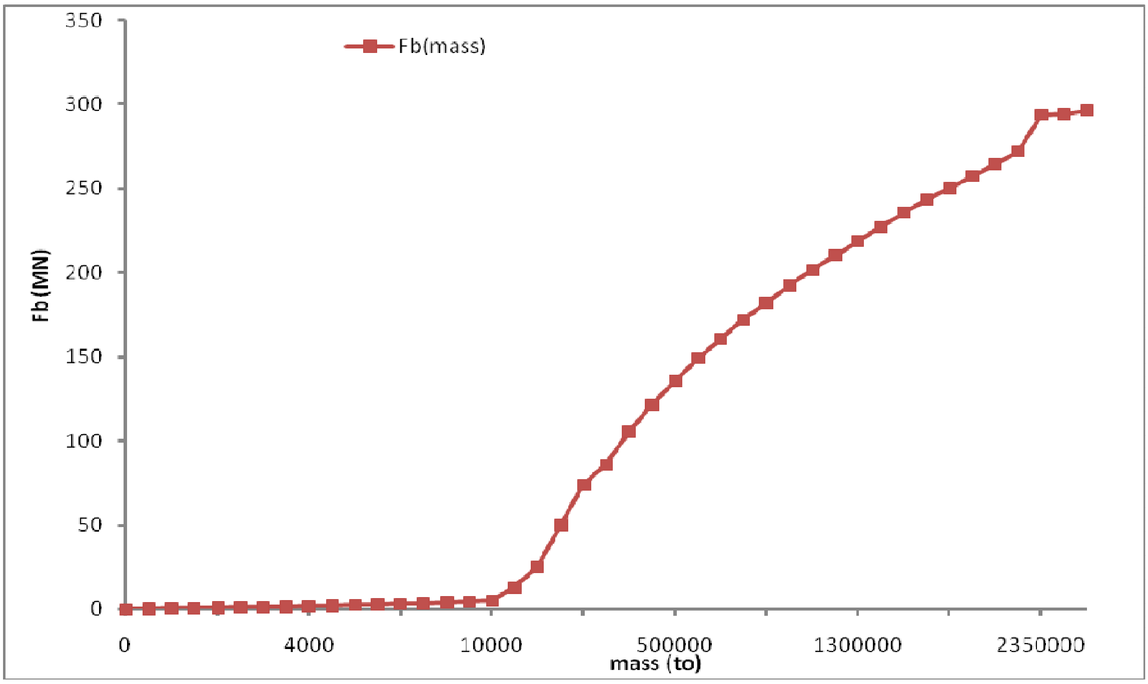

Figure 9. Example for Fb depending on the mass

Fig. 8 illustrates the acceleration S_d depending on the mass. The following mentioned discontinuity points of the curve are related to the discontinuity points of the elastic response spectrum. The curve rises to the point T_B, stays on the same level between T_B and T_C and decends to the point T_D, where it deviates and falls further down. As Fig. 9 shows the horizontal force F_b depending on the mass is continuously rising pitching up at the point T_B. From T_B to T_C it rises with a decreasing inclination and beyond T_D it almost stays on the same level. With increasing mass for simple SDOFs can be shown, that the horizontal force F_b is always rising until T_D whereas the acceleration S_d stays the same between T_B and T_C and is falling beyond T_C.

Tab. 10 compares the results based on the response spectra method with corresponding and maximum acceleration to the results of the modal analysis with varied filling levels of the coal bunkers.

	H (kN)	V (kN)	difference H	%	difference V	%
RSM - max	867	1967	0	0	0	0
RSM - corr	624	1375	261	30	592	30
modal - 100%	479	1039	388	45	929	47
modal - 90%	458	992	409	47	975	50
modal - 75%	426	918	441	51	1049	53
modal - 50%	365	778	502	58	1189	60
modal - 25%	295	609	572	66	1358	69
modal - 10%	247	483	620	71	1484	75

Table 10. Comparison of the results based on response spectra method and modal analysis

As shown within this paper it was possible to reduce the foundation load within the above mentioned structure significantly. It was found that the foundation loads within the simplified model could be reduced in average by 45% compared to the response spectra method with maximum acceleration and by about 15% compared with the response spectra method with corresponding acceleration (Fig. 10), considering that the simplified model gives not the advantages for the modal analysis than a real structure with various loads does. Therefore within the structure of the bunker house the reduction of the foundation load within the earthquake loadcase was 2/3 of the original loads.

6. CONCLUSION

Modelling a structure for seismic design involves decisions on which masses and stiffnesses should be included to maintain an economic but sufficiently safe design.
In the presented example with coal bunkers for a power plant the stiffnesses are defined uniquely by the present steel structure.
Model errors might result from different assessment of the column's bases (pinned or clamped), but with diagonal bracing these errors remain moderate.

As with the coal fill different levels have to be considered which cause variations of the natural frequencies of the dynamic system.
It is assumed generally that seismic loads would decrease if the governing oscillation period is larger than the control period T_C in the response spectrum, since the accelerations will obviously decrease beyond that point.
It seems however that if the frequencies of the structure are altered by additional mass only, the base shear will still increase.
This is due to the fact that the reduction of acceleration with increasing period does not balance the increasing of the mass.

This effect is demonstrated with quantities from a realistic structural analysis of a German power

plant, the numbers were only adjusted to a simplified bunker wall.
It shows that a reduction of the base shear is possible by some 45 % compared to the plateau loads, which is not surprising if the natural period of the structure does not happens to lie within T_B and T_C.
The present bunker wall uses diagonal bracing, which is very stiff, so the period is sufficiently small. Alternatively and with much more effort modal analysis was employed, which is in detail discussed in the paper, and which allowed to reduce the design base shear by another 15 %.

Due to the simplification of the bunker wall for the given example, it showed, that some features of the dynamics of the structure turned out to be different than in the actual design, which could not be discussed in detail for lack of space.
We will report on these effects in a follower paper.

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