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REVIEW EXPERIENCE ON OVERTURNING PREVENTION OF NPP BUILDINGS IN SWITZERLAND

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ABSTRACT

Buildings with high ratios of the building height to the width of their foundations show a great potential for overturning. This problem occurs under large horizontal actions like Safe Shutdown Earthquake (SSE) or Aircraft Impact (AI). There are different concepts to prevent overturning under these actions. During the design process one concept has to be chosen and investigated. Therefore, calculations need to be done, which can be static or dynamic and linear or non-linear. In most concepts the resistance and the activation potential of the foundation soil are important for the overturning stability. Therefore, the definition of nominal values and design values of the soil has to be done carefully. In most of the cases field testing is required due to non-existing data of the soil. Namely the transmission of tensile forces to the underlying soil or rock has to be supported by experimental verification of the system's behaviour.

This paper shows in the first place different possibilities of overturning prevention, which were used at NPP buildings in Switzerland. In the second place a case study of a recently investigated NPP site is presented. Thereby, the potentials and the limitations of static and linear dynamic analysis are discussed and special experiences with non-linear dynamic analysis for earthquake and aircraft scenarios are presented.

Keywords: building, overturning, non-linear, dynamic, aircraft impact, safe shutdown earthquake

1. INTRODUCTION

Several new buildings for auxiliary functions or for spent fuel storage have been built in the last twenty-five years as extensions to the existing five nuclear power plants in Switzerland. Some additional new buildings are currently under construction.

The layout of the existing and operating plants as well as the available space represent geometric restrictions for the dimensions of the new buildings. In several cases the new buildings turned out to have high ratios of the building height to the width of its foundation. The potential of overturning due to extreme design actions from earthquakes or aircraft impacts had to be investigated and prevention concepts were elaborated by the utilities and their engineers.

2. DIFFERENT STRUCTURAL CONCEPTS FOR OVERTURNING PREVENTION

There are different possibilities to prevent the overturning of a building under large horizontal actions like Safe Shutdown Earthquake (SSE) or Aircraft Impact (AI). The chosen concept depends on the local conditions such as available space, geologic conditions, adjacent buildings and so forth, as well as on the intensity of the action. The following structural concepts for overturning prevention were realised.

2.1 Structural connection to an adjacent larger building

The operation building of the NPP Leibstadt (KKL) has been extended for the first time in 1982 ([2]). The extension is a steel construction with concrete composite slabs and the following dimensions: length 55 m, width 8 m, height 27 m (cf. Fig. 1). To prevent overturning due to the design load SSE, it is founded on the operation buildings foundation slab, which was originally extended in prevision of an intended annex. Additionally it is attached to the operation building by 54 anchors for the horizontal stabilization in transverse direction and by 205 shear pins in longitudinal direction.

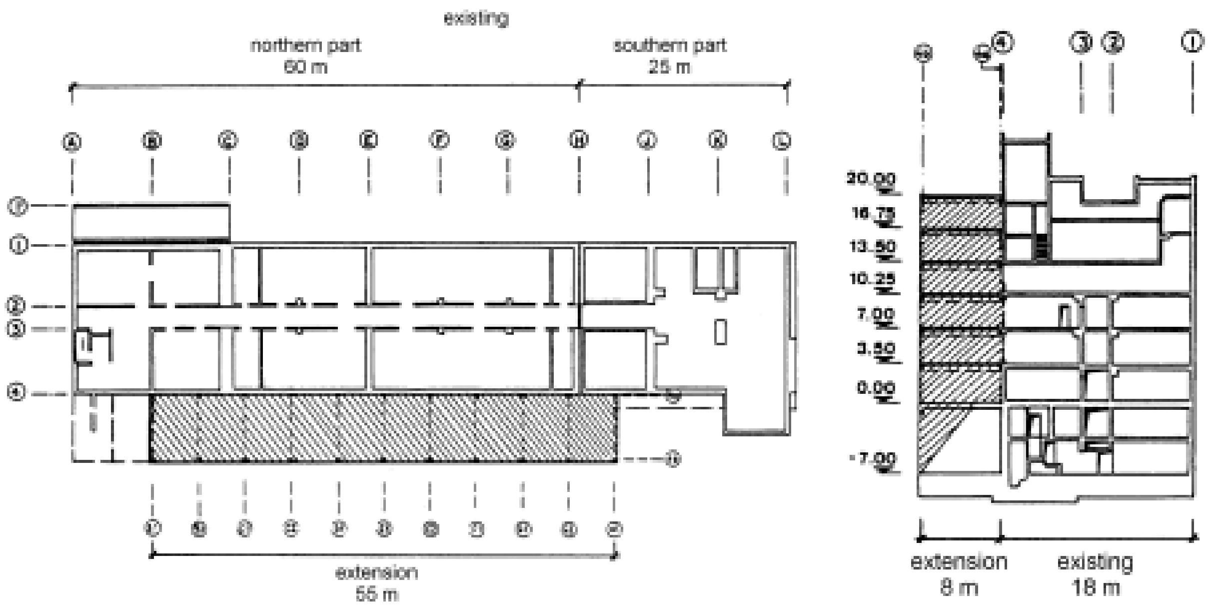

Fig. 1 Overview on the KKL operation building with the first extension ([2])

2.2 Anchoring of the foundation slab to the underlying rock

The extension of the turbine house of the NPP Mühleberg (KKM) was built in the nineties of the last century ([12]). It is a narrow building 49.6 m long, 9.4 m wide and 27.2 m high, which contains maintenance facilities for slightly contaminated waste and fluids (cf. Fig. 2). The extension had to be designed for SSE because of the adjacent turbine house, which must not be affected by the extension under SSE action.

Due to its high slenderness in transverse direction, it had to be secured against overturning. Therefore, the engineers used 107 pre-stressed rock anchors. Each one of them inducing a tensile force of 1.18 MN into the rock (design value containing a safety factor of 1.1 as well as the assumption of 10 % failing anchors). The anchors are placed along the edges of the foundation slab and are between 10 and 12 m long (cf. Fig. 3). The bond length of 4 m results from tension tests of five testing anchors with bond lengths between 3.5 and 7.0 m. The failure load was in each test higher than the required load of 1.58 MN (= 1.33 × 1.18 MN). Several conditions were decisive for the determination of the free length such as activation of enough counterweight, perpetuation of pre-stress after the settlement of the building and limitation of the pressure on the groundwater proofing (located between foundation slab and outer trough). The anchors were pre-stressed with 300 kN.

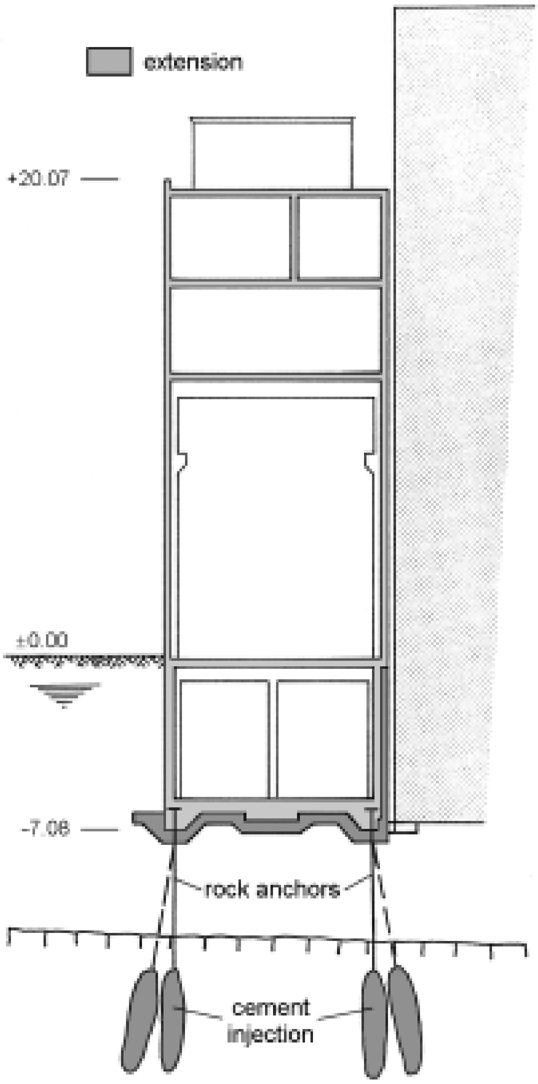

Fig. 2 Cross section through the extension of the KKM turbine building ([12])

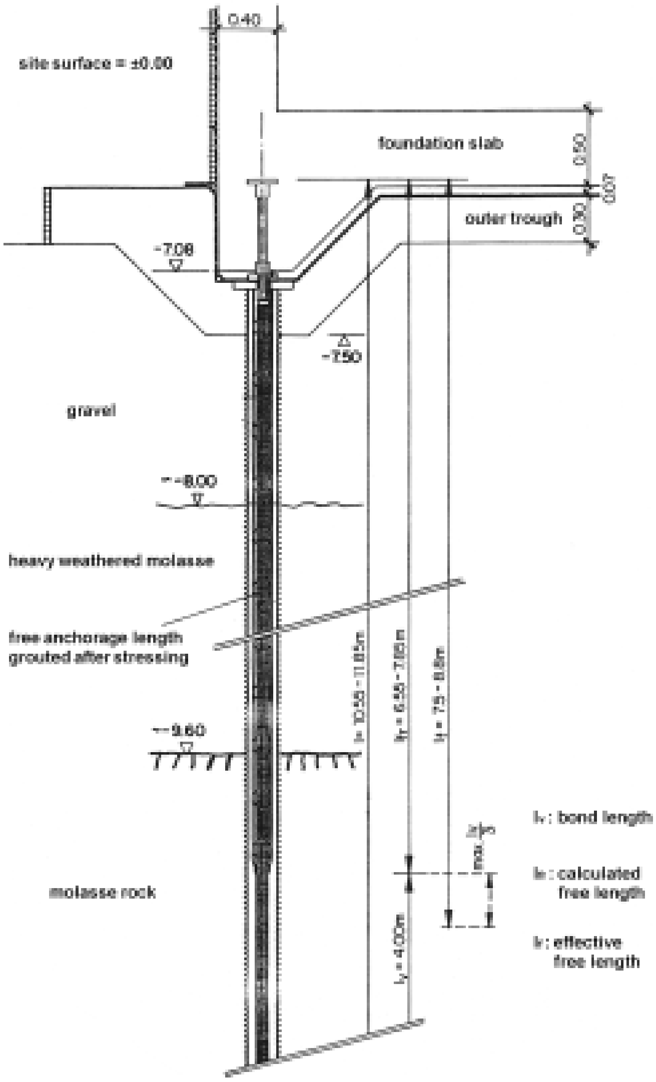

Fig. 3 Detail of a rock anchor ([12])

All installed anchors were tested up to their bearing capacity. About 10 % of them were tested in detail. The tests confirmed the tension tests. To ensure a good durability, the impermeability and corrosion protection were checked by two resistance measurements (before and after stressing) on all installed anchors. The stressing test and the resistance measurements were positive for 93 % of the anchors. The time behaviour can be checked on ten special anchors with accessible anchor heads. This makes force measurements possible whereby the bearing capacity can be controlled anytime during the buildings life.

2.3 Activation of bored piles with tensile capacities

At the NPP site of Gösgen (KKG) there are currently different new buildings under construction: An extension of the auxiliary building (cf. Fig. 4) and a new spent fuel pool building (SFPB) with its cooling towers and operation building (cf. Fig. 5).

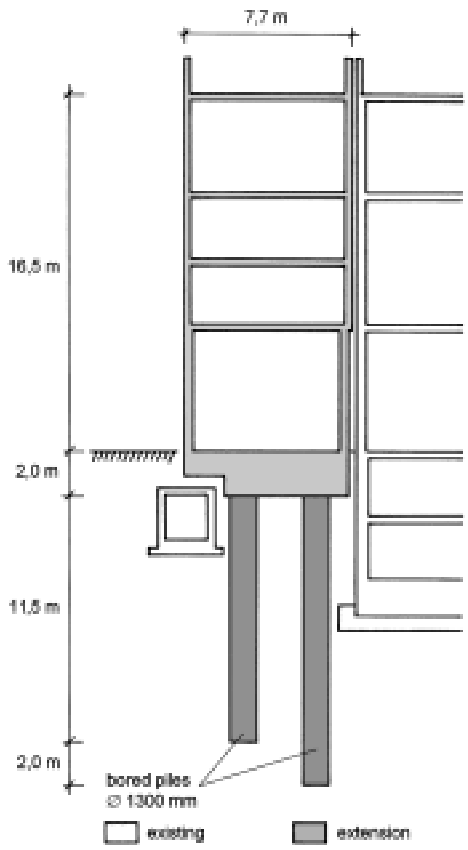

Fig. 4 Cross section through the extension of the KKG auxiliary building

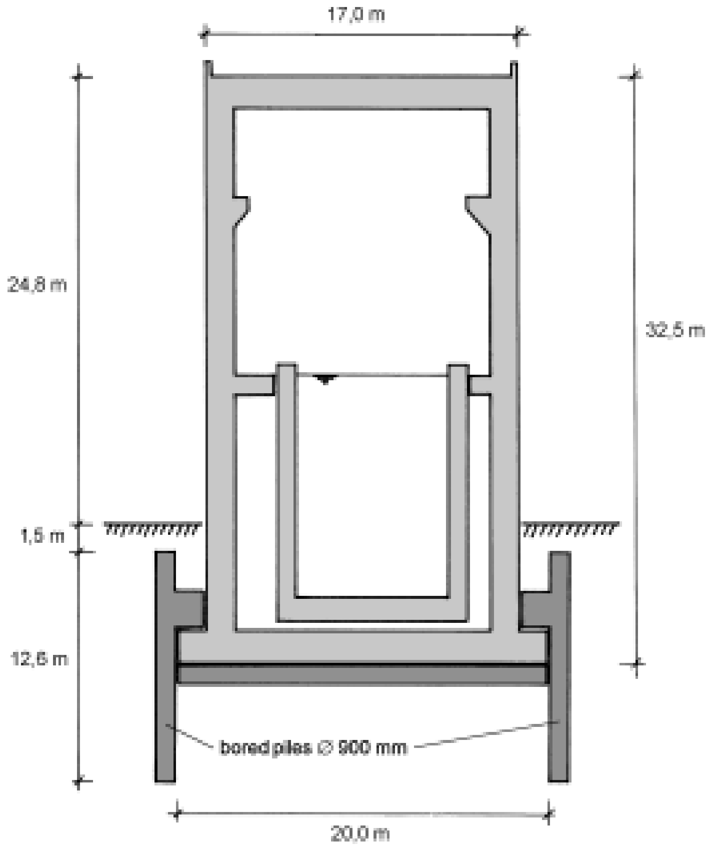

Fig. 5 Cross section through the new KKG spent fuel pool building (SFPB)

The extension of the auxiliary building is 65 m long, 7.7 m wide and 18.5 m high, whereby it is vulnerable to overturning under the design load SSE in transverse direction ([3]). The concept to prevent that failure mode consists in the activation of bored piles with tensile capacities. Because of two existing structures, a parallel running conduit and the foundation slab of the existing auxiliary building, which are situated partly below the extension, the bored piles could not be placed in their most effective way at the edges of the foundation slab. The chosen isolation between the foundation slab and the soil does not allow the transfer of shear forces to the foundation soil. Thus, the horizontal forces have to be conducted through the piles into the soil. Therefore, the piles do not only act as cantilever beams but, in combination with the foundation slab, also as a frame. In addition, the foundation slab acts as a cantilever in transverse direction, because the conduit is not capable to take over extra loads. The different pile lengths result from the different positions of the piles relative to the axis of the extension as well as the different activation potential of the soil due to the basement story of the existing building.

With a length of 35.5 m, a width of 17 m and a height of 32.5 m the new spent fuel pool building does not show a very high ratio of the building height to the width ([8]). Nevertheless, the overturning stability was not sufficient, because of the high horizontal design loads SSE and AI. On this account, a concept was worked out which allows the activation of the bored piles of the excavation retaining wall as a tension element. Therefore, investigations and calculations were carried out. They are presented in the following chapters.

3. OVERTURNING PREVENTION OF THE KKG SPENT FUEL POOL BUILDING

The new KKG spent fuel pool building (SFPB) is introduced in the previous chapter. It consists of 1.5 m to 1.8 m thick concrete enclosing walls, a concrete internal structure for gateway, unloading and handling of the transport container and the spent rod, as well as the storage pool itself which consists of 1.1 m to 1.4 m thick concrete walls and slabs (cf. Fig. 5). The storage pool is separated from other structures by 0.15 m thick joints and is supported by spring elements which prevent shocks on the pool due to SSE or AI.

3.1 Actions

The new KKG spent fuel pool building had to be designed for the extreme actions SSE and AI, which are able to cause overturning. The SSE action at foundation level is defined by the KKG-2002 acceleration response spectrum (cf. Fig. 6). A damping of 5 % for the concrete structure under SSE action was assumed. Synthetic time histories of the ground motion, generated form the acceleration response spectrum using the FE-Programm CASTEM 2001, were used for time history calculations for the review analysis. CASTEM 2001 is a program of the French Atomic Energy Authority CEA (Commissariat à l’Energie Atomique), which is appropriate for non-linear, dynamic calculations. The AI action is defined by the Swiss Federal Nuclear Safety Inspectorate (HSK) (cf. Fig. 6). It is the force-time- history of the impact of a military plane with a mass of 20 Mg and a velocity of 215 m/s on a circular area of 7 m^2.

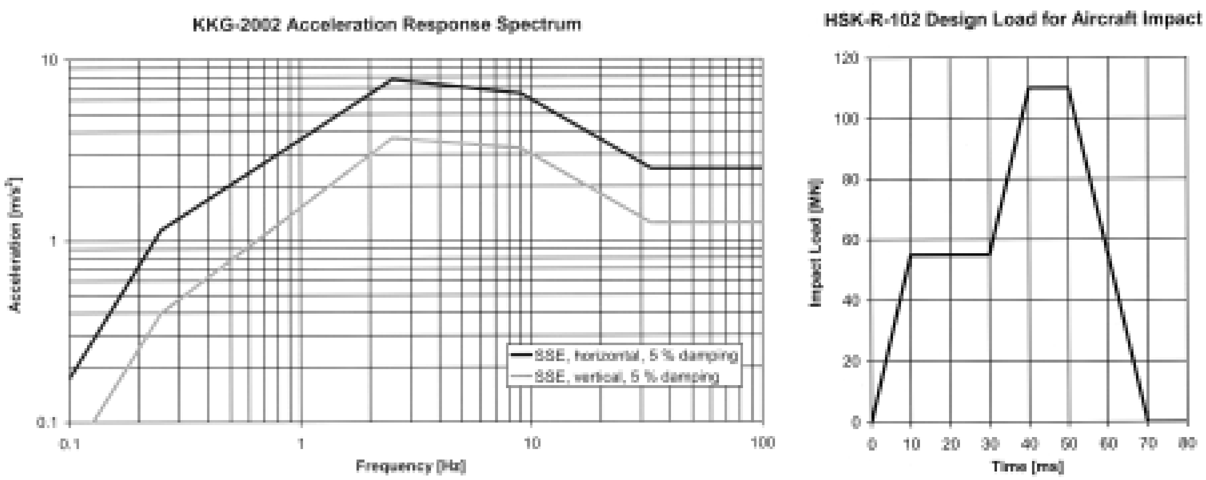

Fig. 6 Actions that cause overturning: SSE ([8]) und Aircraft Impact ([13])

3.2 Field tests to define the tensile capacity of the bored piles

Two tension tests were carried out to determine the tensile capacity of the bored piles (cf. Fig. 7). The existing and decisive foundation soil at the site of the new KKG SFPB is a gravel with a high bearing capacity ([10]). The two experimental piles with a length of 8 m and 12 m (not including the empty drilling length of 8 m from site surface) and a diameter of 0.9 m showed a experimentally defined maximum tension capacity of about 7 MN ([11]). This force was conducted into the piles in the range between 2.5 and 3.5 m below the pile cap by a strand tendon (cf. Fig. 8). The failure of the piles did not occur during the tests, whereby further FE-Simulations were needed to extrapolate to the effective tensile capacities.

Fig. 7 Testing site for determination of the tensile capacity of bored piles ([11])

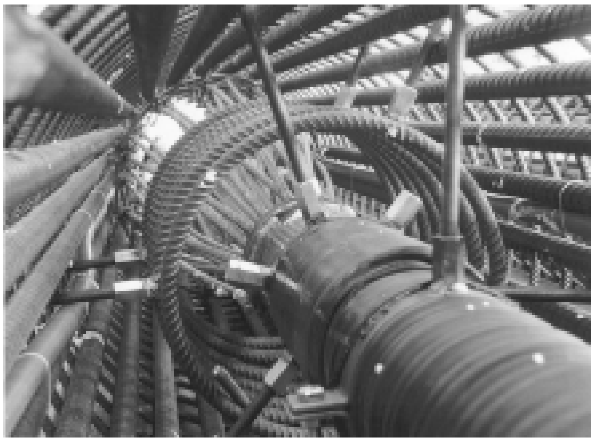

Fig. 8 The reinforcement and the prestressing of the experimental piles ([11])

Effects generated by the self-weight of the pile, the counter bearing and the empty drilling length had to be considered while analysing the experimental data. The following parameters were derived from the tests and further FE-Simulations: the skin friction failure τ_{max} and the horizontal soil pressure coefficient k_h. Thereby, the interpretation of the data and the FE-Simulation was carried out conservatively and shows that both parameters are functions of the pile length, the pile diameter and the distance between the piles. τ_{max} and k_h increase with shorter pile length, smaller pile diameter and larger pile distance. With these relationships the tension capacity of the excavation retaining wall (bored pile wall) alongside the new KKG SFPB was calculated. The elastic stiffness of the bored pile wall was derived from the tension test of the 12 m long test pile. The required force-deformation-relationship for the non-linear overturning calculation is defined by these two values (cf. Fig. 11, springs S3 & S4, including a safety factor of 1.3).

3.3 Different methods to investigate the overturning

3.3.1 Linear static analysis

The simplest way to investigate the overturning is to carry out a linear static analysis with an equivalent force. The safety factor S against overturning can be defined as the ratio of the retaining moment and the overturning moment (cf. equation (1)). In case of the new KKG spent fuel pool building the structural model shown in Figure 9 can be used ([9]). It is based on the following assumptions:

- Acceptable opening of the joint is equal to half of the foundation width.
- Centre of rotation is located in the middle of the foundation slab.
- Triangular stress distribution in the soil.
- Uplift and lateral embedding are neglected.
- Vertical equilibrium according to equation (2).
- Sufficient safety against overturning is given by a safety factor larger than 1.1 (cf. equation (1)).

$$S = M_{retaining} \, / \, M_{overturning} = (T \cdot 9.25m + C \cdot 6.70m) \, / \, M \quad \geq 1.1 \tag{1}$$

$$C = T + N \tag{2}$$

The required values to calculate the safety S were derived as follows: T from the field test (cf. chapter 3.2), N from the self weight of 27'500 Mg (in case of earthquake reduced by the relieving component: the vertical equivalent static force) and M from the horizontal equivalent static force multiplied with its lever-arm relative to the foundation slab. Thereby, the equivalent static forces are defined in Figure 6. In case of the SSE they can be calculated using equation (3). The relevant natural frequencies of the new KKG spent fuel pool building are 3.1 Hz horizontal and 8.9 Hz vertical ([5]). The results of this static analysis are shown in Table 1.

$$F = W \cdot a \tag{3}$$

3.3.2 Dynamic linear time history analysis

A second method to investigate overturning is based on the same model and on the same assumptions as that one discussed in the previous chapter. The difference lies in the calculation of the minimal active axial force N and the overturning moment M using a dynamic analysis, either the response spectrum method (only SSE) or the time history method (SSE and AI). This can be carried out using different structural models like simple two-dimensional models with point masses or more complex three-dimensional models.

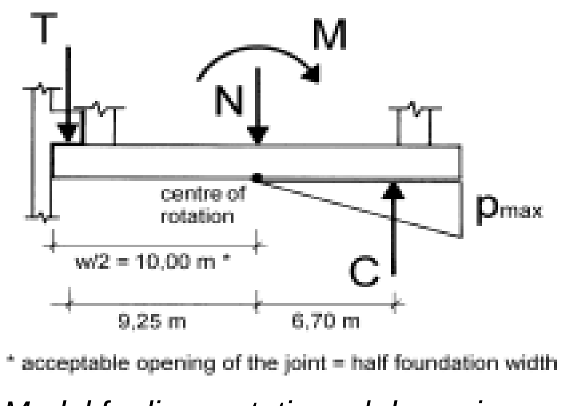

Fig. 9   Model for linear static and dynamic overturning analysis for KKG SFPB

In case of the new KKG spent fuel pool building a detailed three-dimensional model of the building was created by the project engineer using the FE-Program STRUDYN ([5] and [6]). Thereby, the soil structure interaction was represented by six spring elements (three translational springs and three rotational springs). The spring constants were defined by a parameter study of the soil structure interaction ([7]). Finally, the required values of N and M on the foundation bed were calculated using the time history method for SSE and AI. The results are presented in Table 1.

3.3.3 Non-linear dynamic time history analysis

A more complex way to investigate overturning is to consider non-linearities. The building material itself as well as the soil structure interaction can show non-linear behaviour. Again models with different degrees of detailing for the structure, the load and the material can be combined.

In case of the new KKG spent fuel pool building a simple two-dimensional structural model was developed by the project engineer ([4]). The review engineer used nearly the same model to check the analysis ([1]) (cf. Fig. 10). Thereby, a point mass including a transverse mass M_t of 27’500 Mg and a rotational mass M_r of $4.1 \cdot 10^6$ Mgm^2 as well as bending resistant beams represent the building. A rayleigh damping with the coefficients $\alpha = 1.57$ and $\beta = 0.00126$ is implemented. This is consistent with a critical damping of about 5 %.

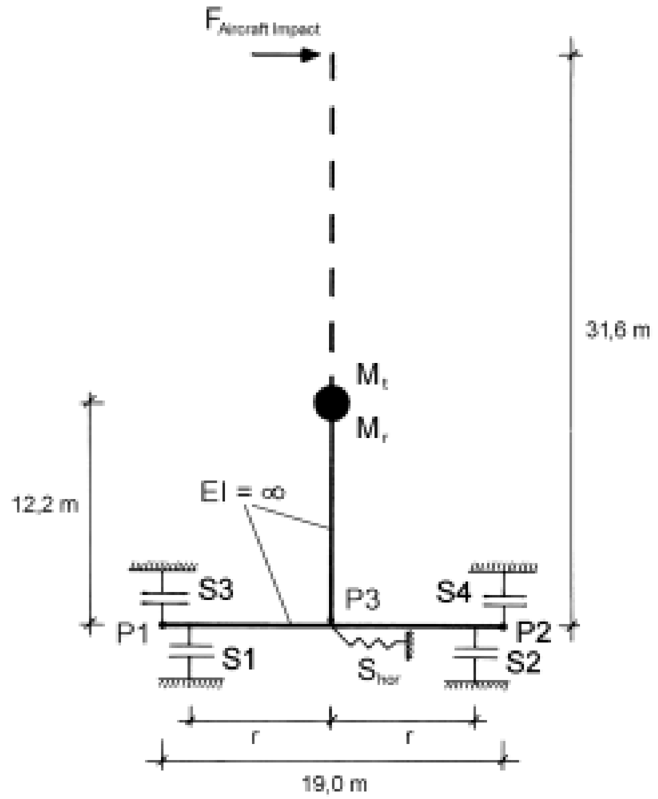

Fig. 10 Model of the KKG SFPB for dynamic non-linear overturning calculation

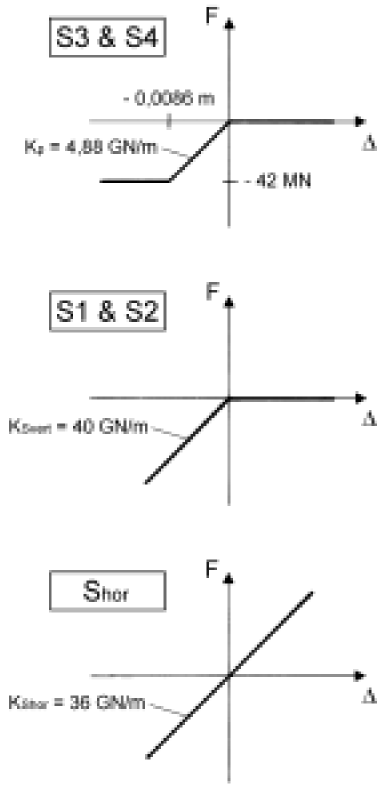

Fig. 11 Material models of the spring elements

Five spring elements generate the foundation soil building interaction. Thereby, the spring elements S1 and S2 represent the foundation soil. They show a non-linearity by only transmitting pressure (cf. Fig.

11). Their spring constant is equal to the vertical bedding modulus under dynamic load of $80 \cdot 10^3$ MN/m and their lever arm r is chosen to simulate the appropriate overturning stiffness of the soil structure system of $5 \cdot 10^6$ MNm/rad ([7]). The excavation retaining wall with its tensile capacity is described by the spring elements S3 and S4. These are implemented in the structural model as pressure elements like S1 and S2. But their non-linearity is characterised by the linear-elastic linear-plastic behaviour shown in Figure 11 which is based on the interpretation of the field tests (cf. chapter 3.2). The spring element S_{hor} represents the horizontal stiffness of the foundation soil of 36 GN/m.

The review analysis was carried out using the FE-Program CASTEM 2001 by applying the time history method with a time step of 1 ms. The SSE actions in vertical and horizontal directions were applied simultaneously. In the case of AI the time history is applied after 1 s.

The Figures 12 to 14 illustrate the results for the SSE. One can see that the piles are activated only a few times during the SSE and that the tensile capacity of the excavation retaining wall is not reached; the wall stays elastic (cf. Table 1). The edge of the foundation slab lifts off by a maximum of 6 mm. This is much less than the acceptable displacement of 90 mm defined as a tenth of the pile diameter. The maximum opening of the joint of the half foundation width is acceptable too (cf. Fig. 18). Therewith, the safety against overturning is guaranteed.

The results for the AI analysis are shown in the Figures 15 to 17. The excavation retaining wall is poorly loaded (cf. Table 1). The vertical displacement (uplift) of the edge of the foundation slab is only 1 mm, so that the gaping joint is less than a tenth of the foundation slab's width (cf. Fig. 18).

Table 1 Comparison of the different calculations

analysis according to chapter		3.3.1		3.3.2		3.3.3 [2)]		3.5
Action		SSE	AI	SSE	AI	SSE	AI	AI
equival. static force F_{hor} / F_{vert}	[MN]	206 / 89	110					
T ($T_{1.1}$)	[MN]	42 (95)	42 (125)	42 (76)	42 (73)	34	5	36
N	[MN]	185	275	228	275		275	275
C ($C_{1.1}$)	[MN]	227 (280)	317 (400)	270 (304)	317 (348)	340	280	365
M	[GNm]	2.51	3.48	2.49	2.74	2.7	2.3	2.0
p_{max} ($p_{max1.1}$) [1)]	[MN/m^2]	1.18 (1.45)	1.65 (2.08)	1.40 (1.58)	1.65 (1.81)	2.2	1.6	1.6
S ($S_{1.1}$)	[-]	**0.76** (1.1)	**0.72** (1.1)	**0.88** (1.1)	**0.92** (1.1)	**> 1.1**	**>> 1.1**	**> 1.1**

1) calculated with the foundation slab's length of 38.5 m*, acceptable is less than 2.5 MN/m^2

2) maximum values during time response, occurring not necessarily at the same time

Note: The calculations in chapter 3.3.2 (linear dynamic) and 3.3.3/3.5 (non-linear dynamic) are done with different time histories of the ground motion; the demands of the latter are slightly smaller.

3.4 Comparison of the different calculations

Table 1 presents the results of the different overturning calculations. As expected, the linear static analysis using an equivalent static force shows the lowest safety against overturning; with only 0.7 the safety is not sufficient. This method is very conservative and can be used to clarify, if a building has a great potential for overturning. The linear dynamic analysis using time histories shows a higher, but still insufficient safety of about 0.9. This method allows a more realistic calculation of the action in the foundation bed and, therewith, a better estimation of the safety against overturning. Finally, the non-linear dynamic analysis using time histories shows the required safety of the new KKG spent fuel pool building. Thereby, the safety is not determined by the ratio of the retaining moment to the overturning moment, but by failure conditions of the structure and the foundation soil. To get a sufficient safety against overturning with the linear calculations, the expensive installation of longer piles in order to increase the tensile capacity of the excavation retaining wall, would have been necessary. This has been prevented due to the more sophisticated non-linear dynamic analysis.

The dynamic time history analysis using a model which represents the whole system (structure and soil structure interaction) and a force and failure criteria in order to determine the safety factor against overturning shows, that in the case of extreme short-time action impulse their exposure time does not suffice to generate the much higher effect calculated statically. Due to the inertia of the system the reaction of the system is delayed and, hence, the demand on the structure is reduced. A next step in overturning investigations could be the consideration of the building’s energy dissipation itself.

3.5 Special experience with the non-linear dynamic analysis

In the case of the non-linear dynamic analysis the influence of different synthetic time histories of the ground motion (SSE, all spectrum conformable to Fig. 6) was investigated using the model of Figure 10. The results show that different combinations of horizontal and vertical time histories create a variation of about ± 15 % of the overturning moments and the horizontal displacements. The variation of the vertical displacement is much bigger due to synchrony-effects of the maximum horizontal action and vertical unloading during the time history. It varies between ± 50 %.

Figure 19 to 21 as well as the right part of Figure 18 show the results of an AI analysis carried out according to chapter 3.3.3 but with a different soil structure interaction model (modified model): for the vertical bedding modulus under dynamic load a much higher value of $200 \cdot 10^3$ MN/m is chosen and the lever arm r is chosen smaller, so that the overturning stiffness of the soil structure system stays unchanged by $5 \cdot 10^6$ MNm/rad. This change strongly affects the behaviour of the building. The lift off and the activation of the excavation retaining wall is increasing due to the smaller settlement under dead load considering the unchanged overturning stiffness (cf. Table 1).

4. CONCLUSIONS

The following lessons should be learned from the presented case studies: The static analysis may be useful for a first and conservative estimate. As long as it provides sufficient safety margins there is no need for further and more sophisticated analyses. However, if the static analysis fails to verify the specified safety margins, the further investigation with dynamic analyses – linear and/or non-linear – is strongly recommended. With these more realistic approaches of the complex uplift and overturning behaviour the retaining structural elements are dimensioned realistically and in a cost-effective way. But thereby, the definition of the model parameters like overturning stiffness, vertical bedding modulus, time history of the ground motion and so forth have to be done carefully. Technical solutions which depend on tension elements to be activated during extreme events – for retaining the uplifting building – must be qualified with supporting field tests complementing the analysis.

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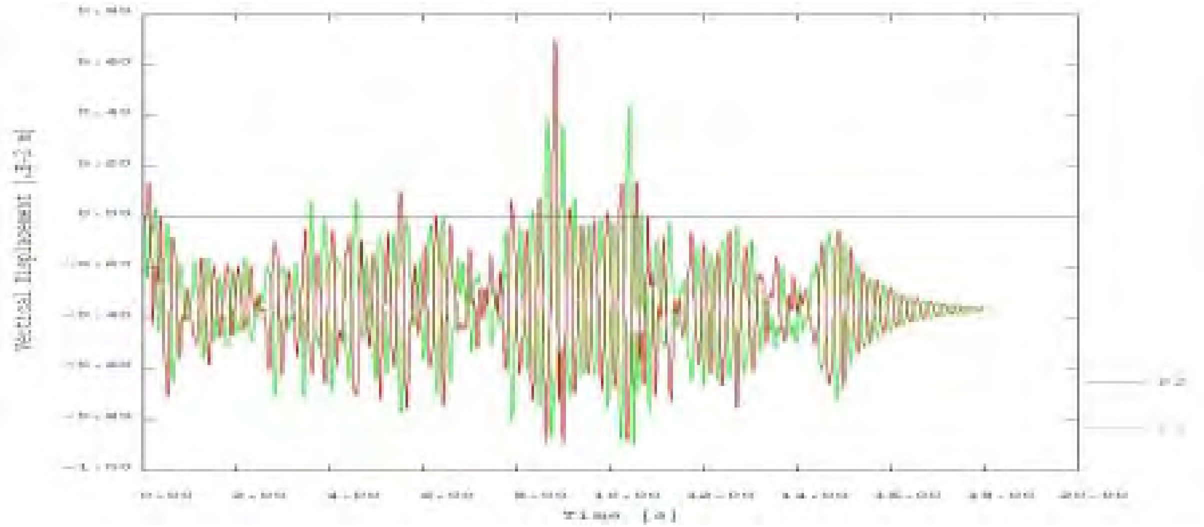

Fig. 12 Time response of the foundation slab's vertical displacements under self-weight and SSE

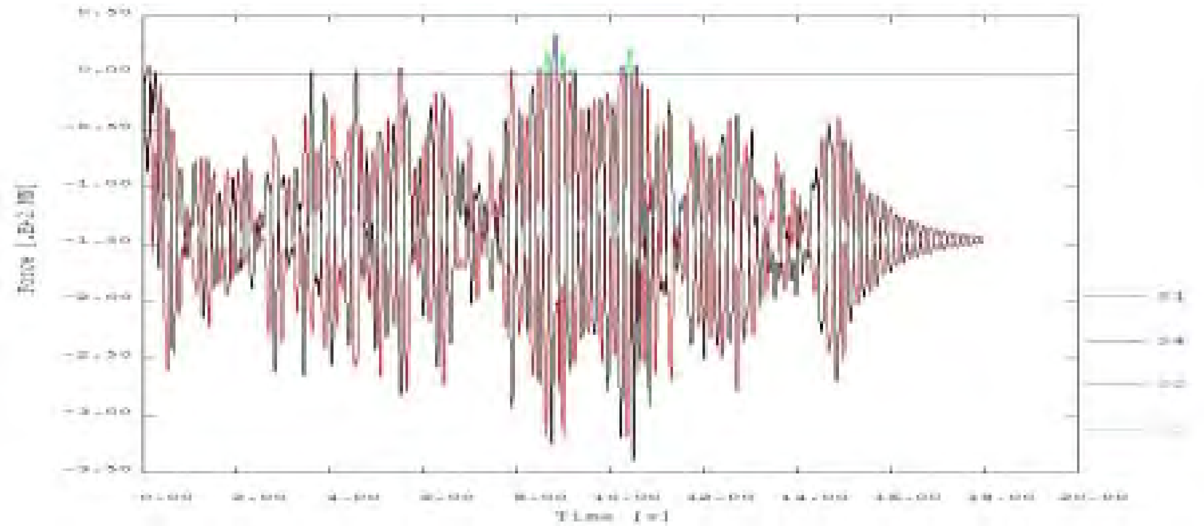

Fig. 13 Time response of the foundation forces under self-weight and SSE

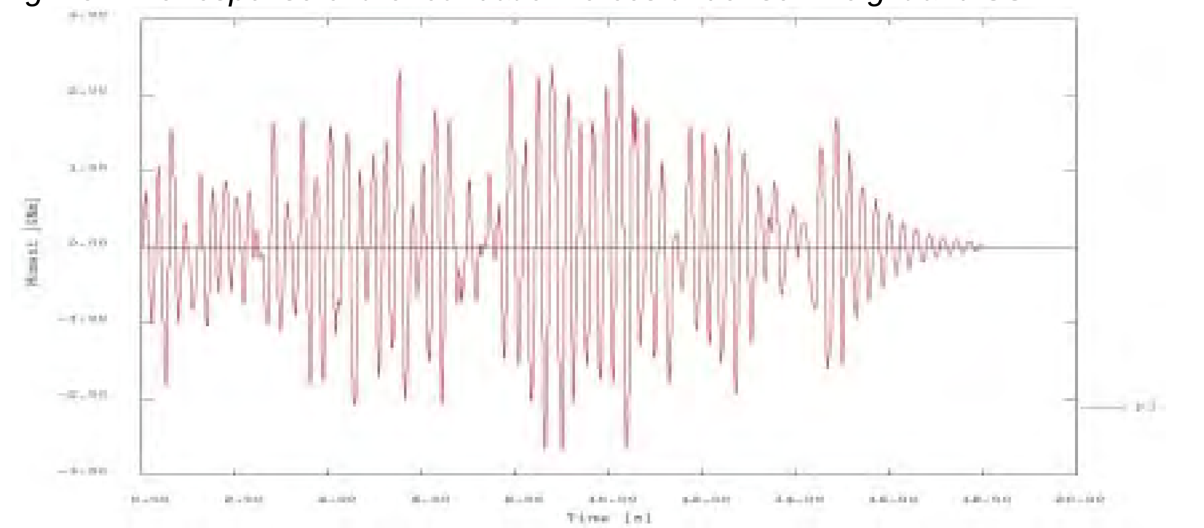

Fig. 14 Time response of the overturning moment under self-weight and SSE

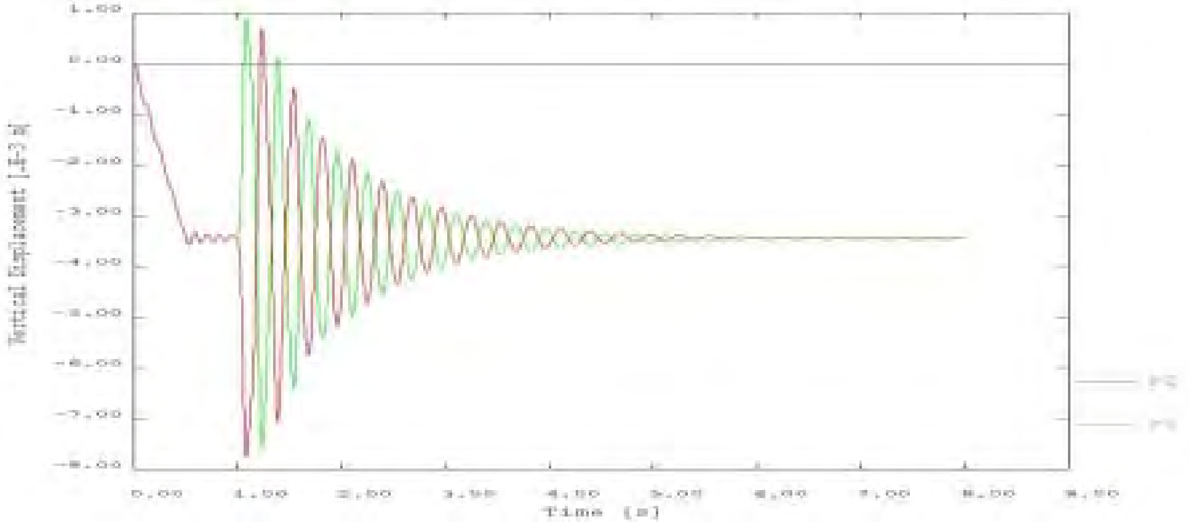

Fig. 15 Time response of the foundation slab’s vertical displacements under self-weight and AI

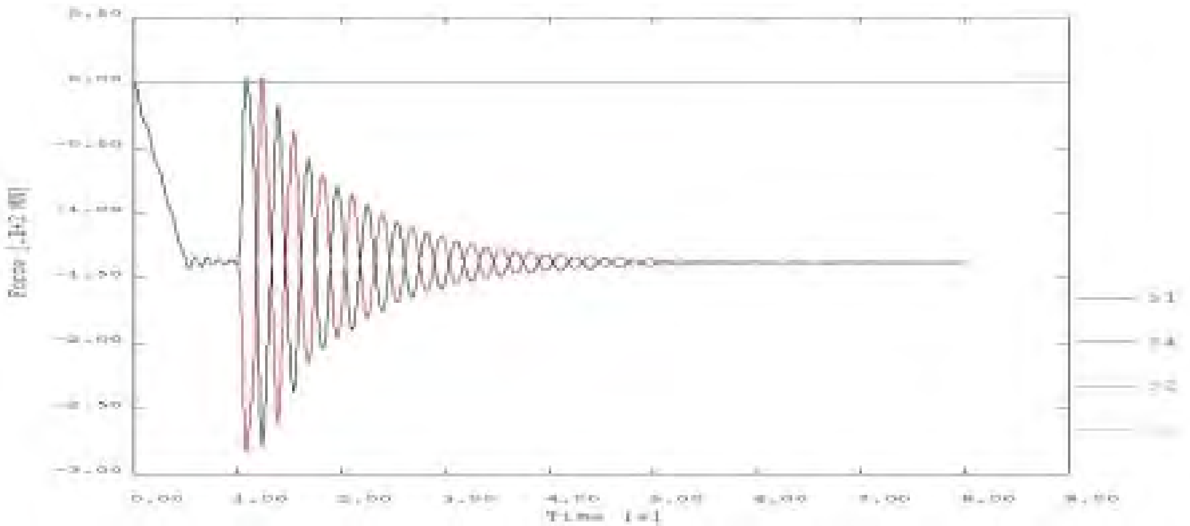

Fig. 16 Time response of the foundation forces under self-weight and AI

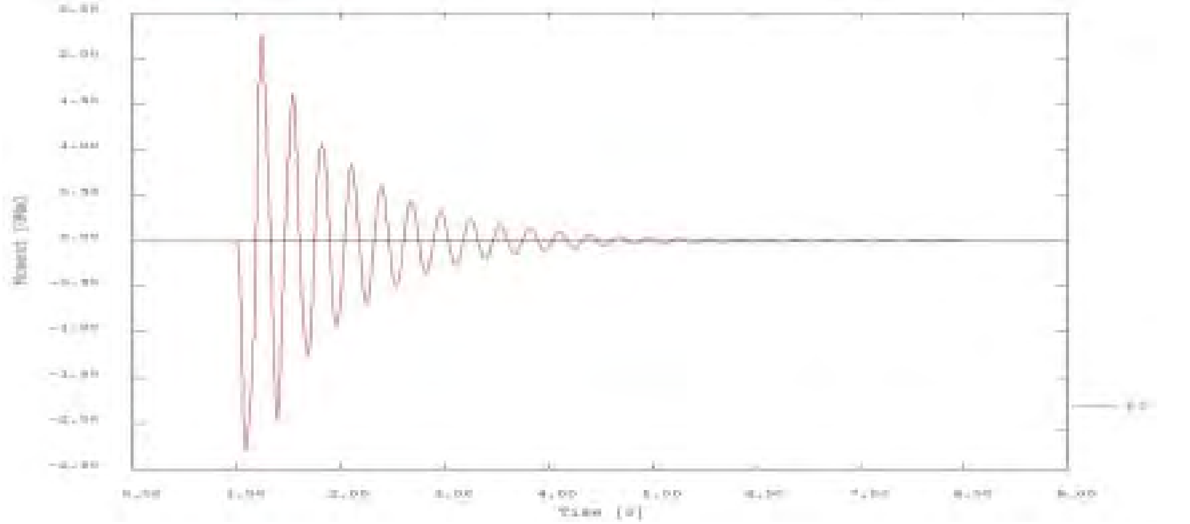

Fig. 17 Time response of the overturning moment under self-weight and AI

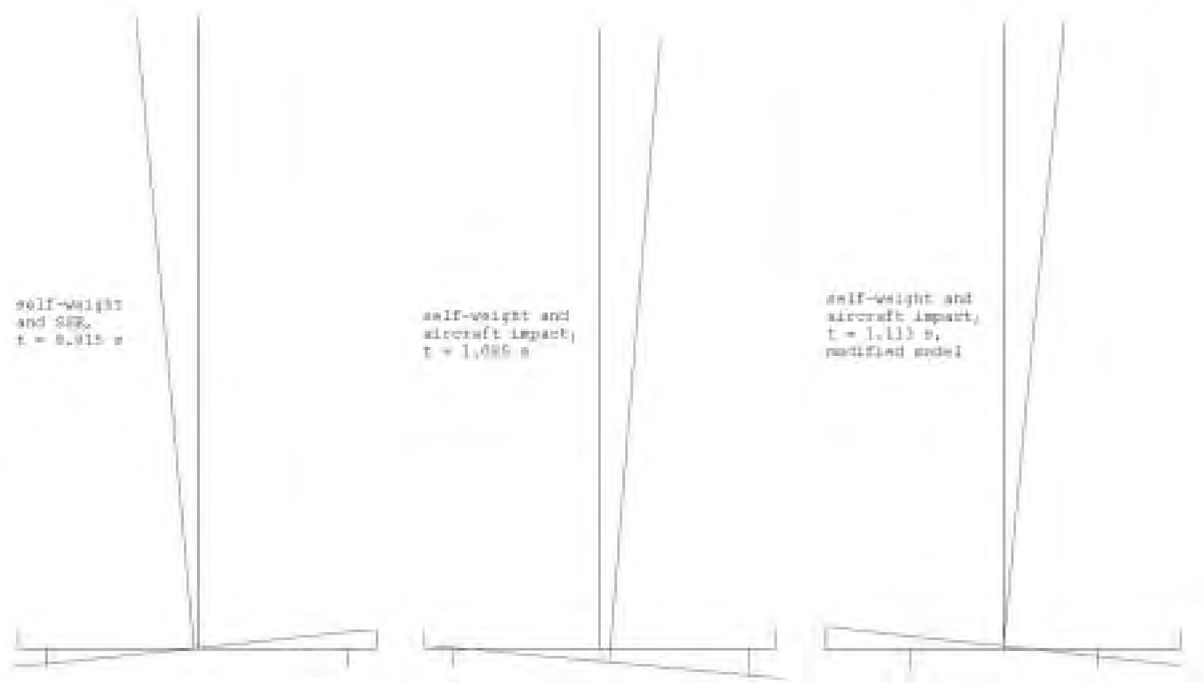

Fig. 18 States of the maximal overturning (excessive to visualise the opening of the joint)

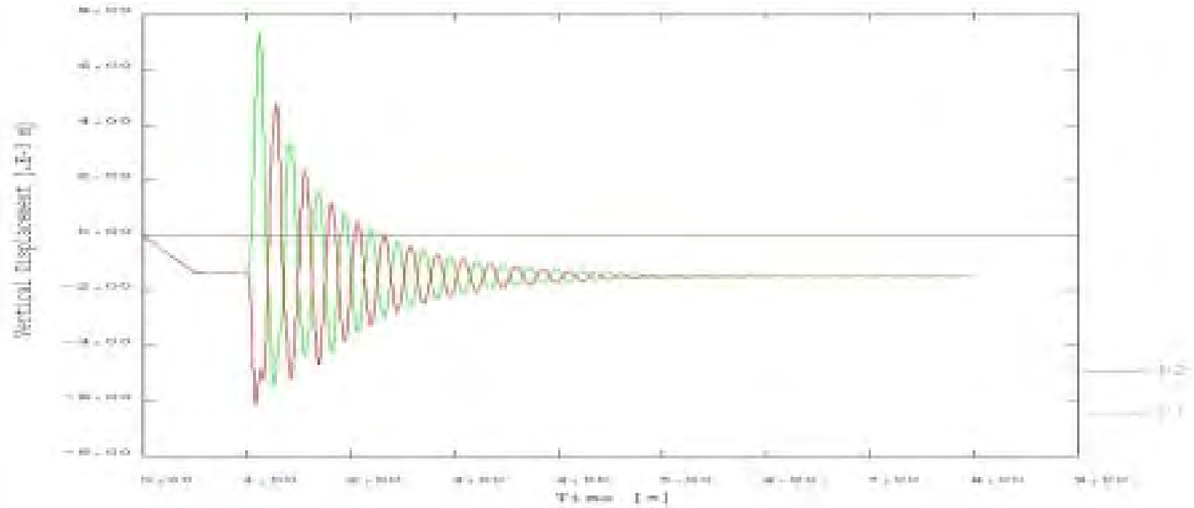

Fig. 19 Time response of the vertical displacements under self-weight and AI, modified model

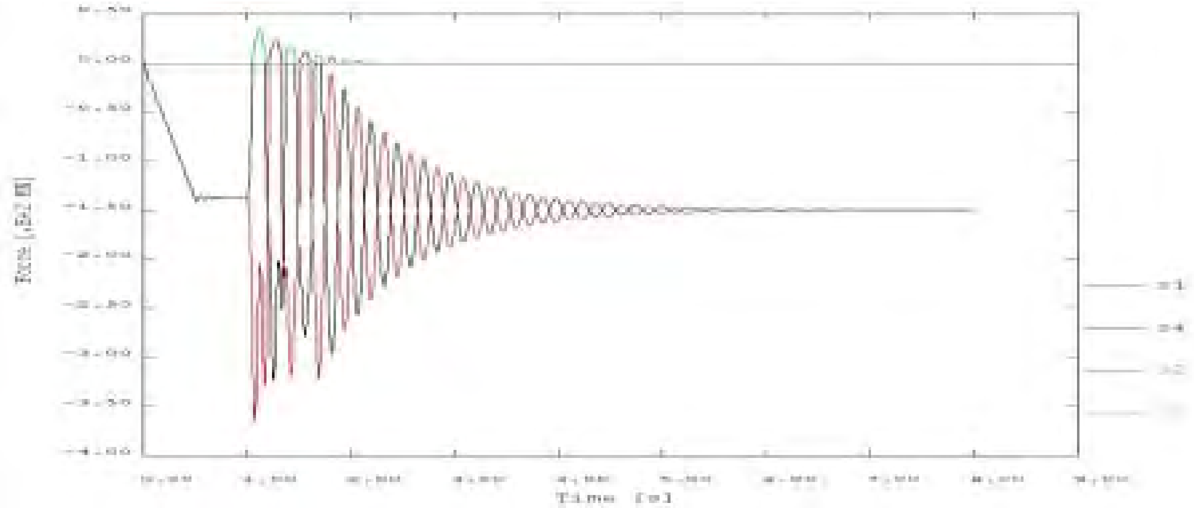

Fig. 20 Time response of the foundation forces under self-weight and AI, modified model

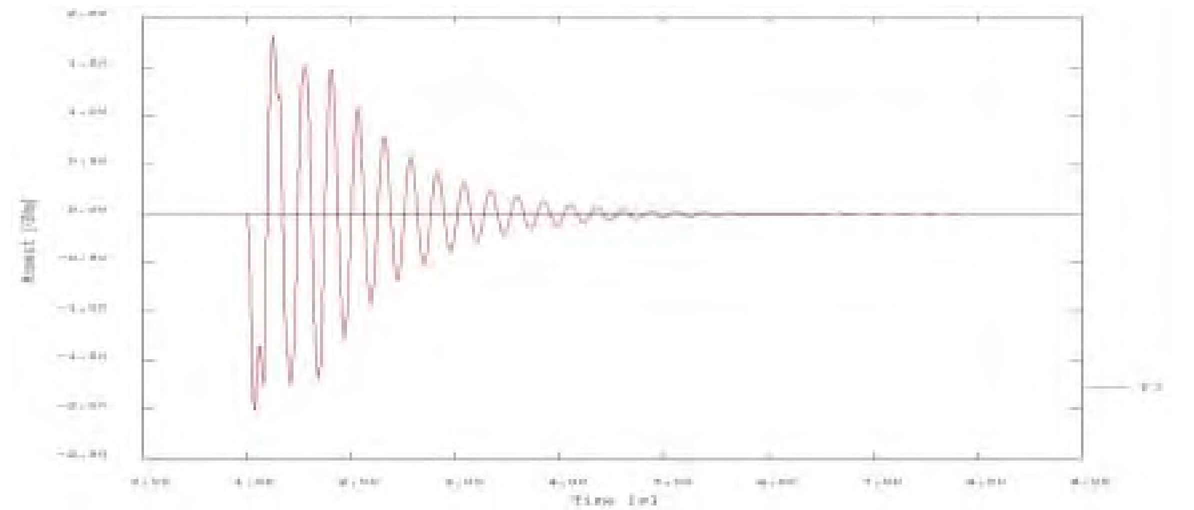

Fig. 21 Time response of the overturning moment under self-weight and AI, modified model

