Slender Vessels – Non-linear Analysis of Stability under the Impact of Dynamic Loads

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ABSTRACT

Slender vessels or tanks may stand on their supporting ground without anchoring or with incomplete anchoring. Under the impact of horizontal dynamic loads a simplified proof of stability by static calculation is only possible up to a low load level. Friction at the ground, bearing capacity of anchors and the ratio of horizontal and vertical accelerations in connection with distribution of mass and geometry of the vessel are the important parameters.

If the loads exceed the low level realistic linear or non-linear dynamic analyses may be carried out to prove the stability of the initial system or to find the limits in geometry and mass distribution or to point out requested measures for stability within prescribed safety margins.

Two examples will demonstrate the procedure of proof of stability:

- Vessel stack on concrete plate without anchoring – earthquake loading.
- Pressure tanks of a safety relevant system on concrete plate with partial anchoring – base excitation according to an external event and impact load due to pipe rupture.

At an interim depository in the hall for radioactive waste vessels shall be stored up to 4 one upon the other. The lower vessels are put on the concrete floor without any anchoring and with defined distances in horizontal direction to the next ones. Exactly centred, the upper vessels are put on the lower ones by means of stacking aids, but also without vertical anchoring. For the site of the depository an earthquake is defined the level of which makes dynamic analyses necessary. For one vertical vessel stack a Finite Element (FE)-model is established which contains rigid vessels one upon the other, springs and friction-gap-elements between lower vessel and ground slab as well as vertical gap elements and horizontal springs between the vessels. With earthquake time history input on this FE-model the stability of two vessels one upon the other can be proved. The necessary horizontal distance between the vessel piles is an additional result.

Tanks of a safety relevant system of a nuclear power plant (NPP) stand on the concrete base plate of the reactor building. They are partially anchored with rag bolts. For the different dynamic load cases base motion due to an external event (EE) respectively rupture of an attached pressure pipe (RPP), rocking respectively sliding must be analysed. A FE-model is generated which regards mass and stiffness of the different parts of the tank which are inner tank, outer pressure vessel and burst protection facility as well as the complex support construction. Non-linear effects between the base plate and the concrete plate such as friction, gap and plastifying of the rag bolts are taken into account. The application of different time histories of accelerations (EE) resp. hydraulic forces (RPP) on the FE-models in non-linear dynamic analyses shows sufficient safety against rocking and sliding. For both examples no measures are necessary.

INTRODUCTION

For different NPPs the stability of components under the impact of different dynamic loads has to be investigated. The components are similar with respect to their slenderness. They differ in function, applied loading and anchoring for stability.

Because of the support of the structures with no or only partial anchoring and in combination with the high level of dynamic loads in the horizontal and vertical directions neither simplified static calculations nor linear dynamic analyses are appropriate for the proof of stability. Thus the investigations are carried out by non-linear FE-analyses.

Within the past years at the sites of German NPPs interim depositories for the storage of spent fuels or of radioactive waste were erected or existing buildings have been adapted for this purpose. Containers or vessels simply stacked on concrete plates are used as storage facilities. There is no anchoring of the lowest vessel to the concrete plate and - in the worst case - even no anchoring between the vessels among each other. Within the concept-finding phase and for final design the stability of these stacks had to be investigated under the impact of earthquake loading. Due to the different seismic load level for each NPP site different analyses were carried out, from applying quasi-static loads up to non-linear dynamic calculations. Also different arrangements of the facilities were analysed which are stacks from 2 up to 4 one above the other, total height 6.22 m. Vessels with outer diameter / height = 1060 mm /1510 mm and max. 2340 kg mass turned out to be the most sensitive facilities. For a site with relatively low seismicity a stack up to 3 vessels (fixed among each other in horizontal direction) could be proved by a simple static analysis. The proof of stability of a stack with 4 vessels could be furnished with a non-linear dynamic analysis. The same was realised for a respective stack with only 2 vessels for a site and erection place with significantly higher seismic excitation. This most interesting example will be discussed in the following.

Slender pressure tanks are part of a safety relevant piping system. One tank system consists of an inner tank, the outer pressure vessel and a burst protection facility as well as of the complex support construction which is anchored to the concrete base plate by 10 rag bolts. The rag bolts transmit shear forces, but no significant tension forces. The dimensions of a tank system are about outer diameter / height = 2.5 m / 7 m; the total mass is about 110 t. Beside dynamic operation loads the dynamic loads according to the decisive EE blast wave (BW) and the inner faulted condition (RPP) must be regarded. Simple static stability checks show the necessity of detailed analysis procedure which will be presented in the following.

VESSEL STACK

Within the scope of this presentation a stack of two vessels with total mass of 16 t is discussed. The contact between the two vessels is supplied with a stacking aid which transmits horizontal forces as well as vertical pressure forces only. Between the lower vessel and the concrete plate there is no anchoring; forces are exclusively transmitted by normal forces in vertical direction and by friction forces in horizontal direction. For the non-linear earthquake-analysis a FE-model is developed and base excitation is applied to this model realistically.

FE-Model

The compact casting vessels with 1500 mm height, circular cross section d_0 = 1060 mm and with a diameter of contact area d_0 = 810 mm are modelled by stiff volume elements. For the contact conditions between the two vessels vertical compression only springs and simple horizontal springs are used. Between the lower vessel and the concrete base plate vertical gap-elements are introduced with non-linear spring-friction capability tangential to the horizontal contact plane (see figs. 1 and 2). The lower ends of the gap-elements around the edge line of the contact area to the concrete plate are connected via springs (appropriate for the stiffness of the concrete plate) with a rigid system to the centre point of the contact area. This central node is fixed in all 6 degrees of freedom. The earthquake loading is introduced at this central node by acceleration time histories simultaneously for all three translational degrees of freedom (see fig. 3).

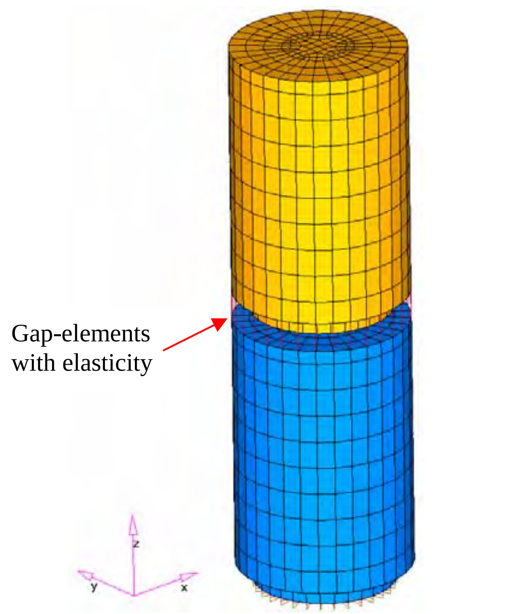


Fig. 1: Calculation model of vessel stack, overview

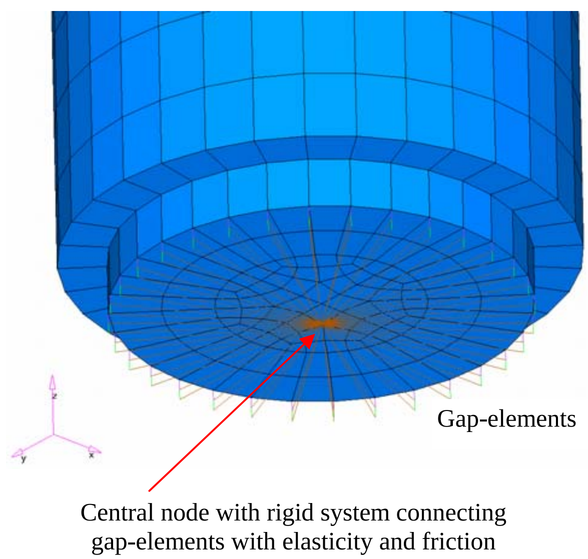


Fig. 2: Calculation model, detail at the base

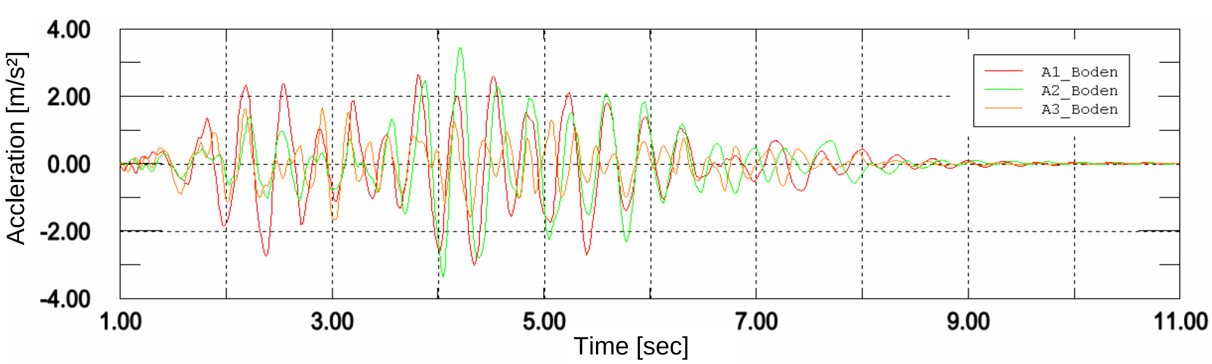


Fig. 3: Earthquake base excitation at centre node, acceleration time history

Calculation Results

Displacements relative to the base plate at different points of the stack, maximum rocking angle of the lower vessel, reaction forces at the centre point on the base plate and maximum gap forces at the base are calculated responses. The behaviour of the two vessels under the impact of the earthquake with 10 s duration time is summarised in a video presentation. It is characterised by a tumbling movement with always complete contact between the two vessels. The vessel stack does not overturn. The displacements of the centre point of the lower vessel are shown in fig. 4 with remaining horizontal portions. Due to small damping the vertical portion slowly decreases to zero.

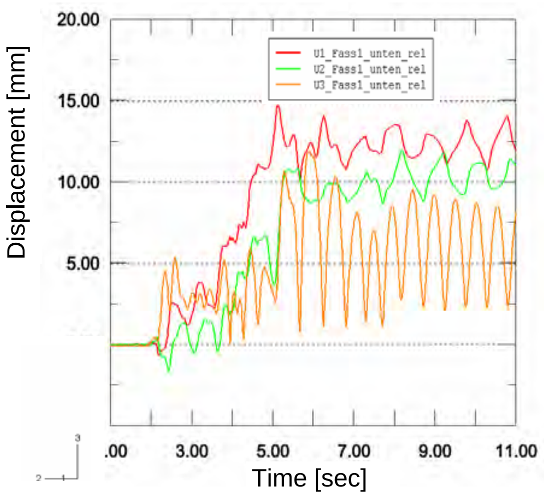


Fig. 4: Displacement of lower stack plane relative to the base plate

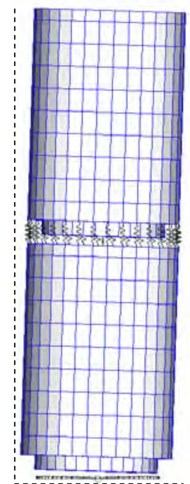


Fig. 5: Maximum displacement (inclination)

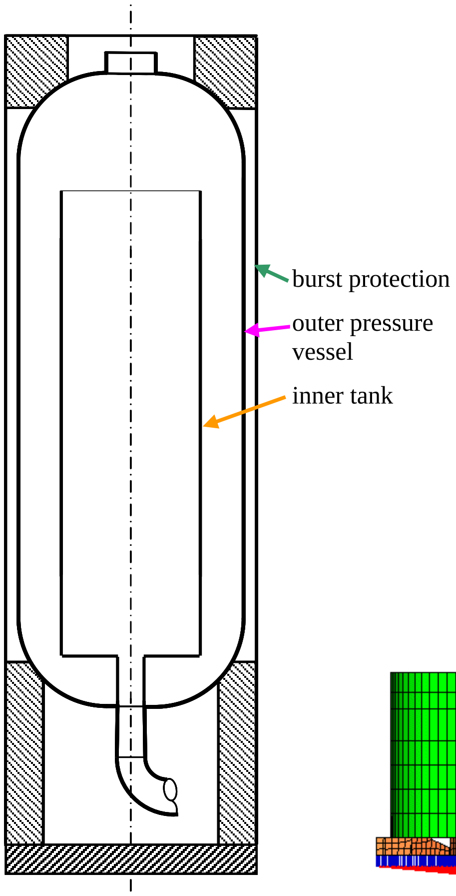
The resultant vertical reaction force varies around the reaction force due to dead load with a maximum shock value downward of about 2-times dead load. The maximum horizontal reaction forces are 0.3-times dead load, thus no sliding appears. At the time with maximum displacements only 4 gap-elements remain closed, but at the time with maximum vertical reaction force clearly more gaps are closed. Force transmission between lower vessel and base plate is possible without any problem. For the technical preparation of a stacking plan the maximum calculated horizontal displacements of 150 mm are relevant. Parameter studies show that the maximum response depends on different combinations of maximum amplitude and frequency content of the excitation time history as well as the diameter of the contact area with the base plate. Higher system damping is the only parameter which solely decreases the system response.

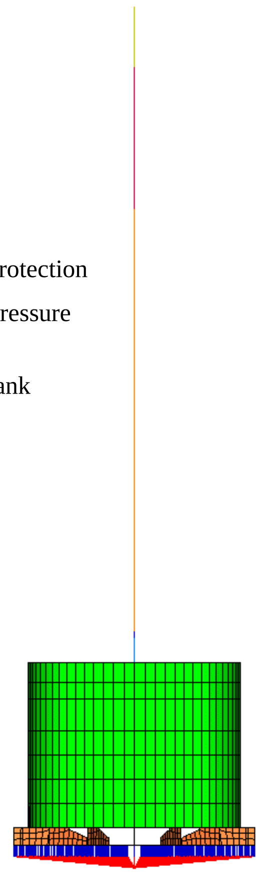
PRESSURE TANK

The environment of the slender tank system (fig. 6a) is protected against burst of the pressure retaining tank wall by a burst protection facility, but not completely against overturning or major sliding movements of the tank system at all. The rag bolts between support construction and concrete base plate are not appropriate for transmission of significant tension forces. Thus for the dynamic faulted load conditions EE and RPP (both service level D) the safety margin for overturning and for sliding must be evaluated. EE loading acts as base excitation, RPP acts shock like as a horizontal single force time history beneath the lower tank head.

FE-Model

The primary aim of the analyses is to prove safe load transmission between support construction and concrete base plate without overturning or infinite sliding. Because of no additional horizontal support of the tank system along the total height the only means for force transmission are pressure with combined friction between base plate of the bearing construction and concrete plate and shear of the rag bolts. All other parts and connecting elements of the system are over-dimensioned. For the purpose of calculations a Finite-Element-Model (FEM) is developed. The beam like parts of the system, which are inner tank, outer pressure vessel and burst protection facility are modelled with beam elements, whereas the complex support construction with plates and ribs is introduced into the model as plate elements (complete model and details see figs. 6b, 6c). The stresses in these plate elements will be calculated. They are assessed to be representative for the strength of the tank structure under the impact of the discussed dynamic loads EE and RPP. For realistic simulation of the force transmission between the base plate of the supporting structure und the concrete base plate gap-elements are put between the steel base plate and the concrete base plate which is idealised with a rigid system. The central node of the rigid system lies under the tank axis and will be excited for simulation of the EE load case. For the simulation of the RPP with about 4-times greater horizontal forces the combined force transmission by friction and by resistance of the rag bolts is regarded. For this purpose the gap-elements are used with friction option (friction coefficient μ = 0.25) and the 10 bolt positions of the steel base plate are connected by a rigid system the centre of which is supported by a horizontal non-linear spring. The characteristic of the spring is 1 mm clearance between bolt and plate, linear stiffness of the bolts embedded in concrete up to yield point 430 kN, linear stiffness up to ultimate load 750 kN (see fig. 15).





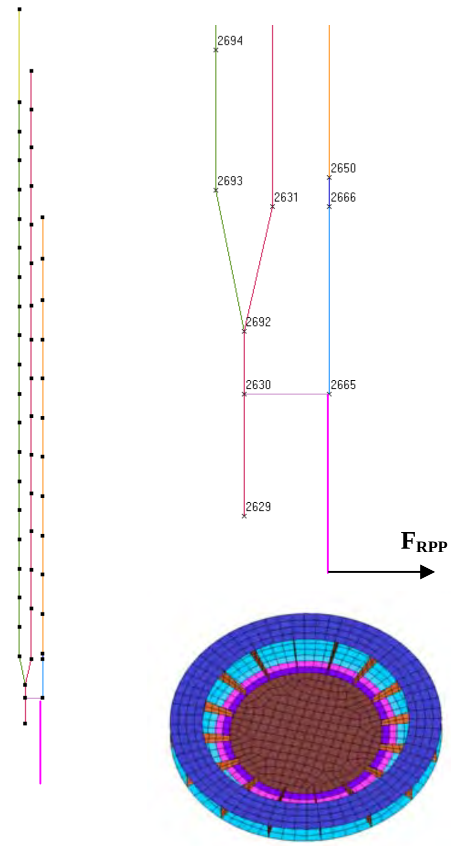
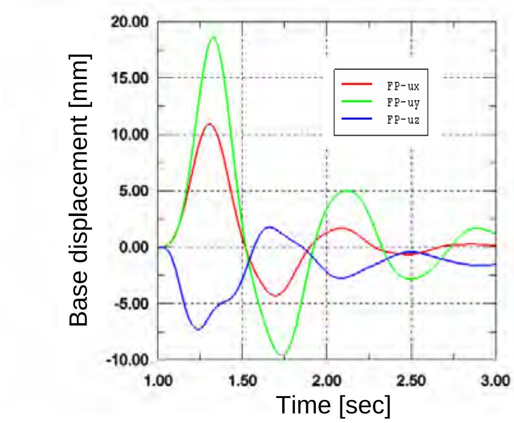


Fig. 6a: Tank system Fig. 6b: Complete FE-Model Fig. 6c: Model details

The load due to the external event serves as input to the model at the base (central node of the rigid system) as calculated with a global FE-Model of the building. Displacement time histories synchronously acting in the three directions of space are the loading functions (see fig. 7a). The maximum input displacements are correlated with maximum accelerations of 2 m/s².



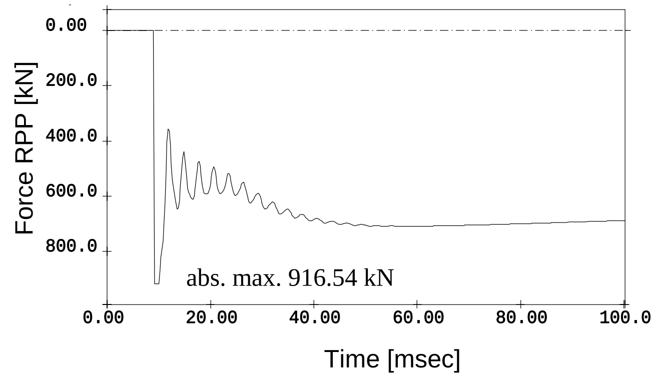


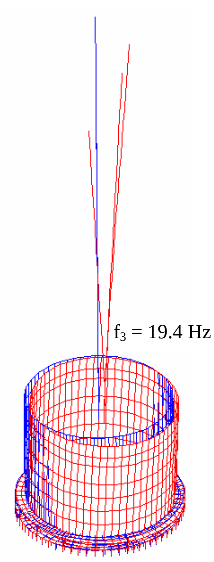
Fig. 7a: Loading functions due to EE

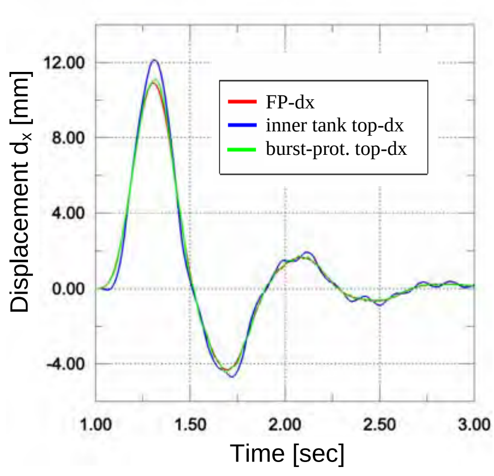
Fig. 7b: Hydraulic force due to RPP

The force time history due to rupture of the pipe in the lower head of the pressure vessel was calculated with a fluid dynamic analysis. According to the assumed rupture of the first welding in the horizontal part of the pipe the force acts on the respective nozzle in horizontal direction (position see fig. 6, time history see fig. 7b).

Calculation Results for load cases EE and RPR

Basic information about the dynamic behaviour of the tank system itself is taken from analysis of the free vibrations of the tank system with the three substructures (beams) on the supporting structure rigidly fixed at the steel base plate. The lowest two frequencies f_1 = 6.7 Hz and f_2 = 8.5 Hz are related to vibrations of the inner tank flexibly supported at the plane bottom by the lower nozzle of the pressure vessel (fig. 6a). Individual movements of the single beams appear at f_3 = 19.4 Hz (fig. 8) and at higher frequencies $f_i \geq 47.9$ Hz.





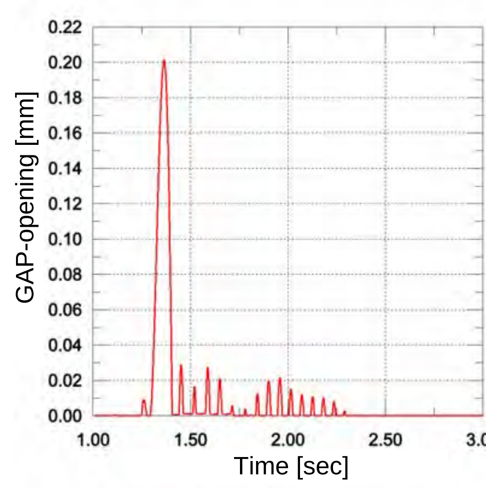


Fig. 8: Mode 3

Fig. 9: Displacement response to EE

Fig. 10: GAP-opening during EE

The displacement response to the load case EE (fig. 9, x-direction for example) shows that with respect to the input (FP-dx, see also FP-ux of fig. 7a) there is only small amplification at top of the burst protection facility and of the inner tank. The maximum appears at the inner tank the time history of which shows small superimposed oscillations with the basic frequency of f_1 = 6.7 Hz. Together with time history of the gap-opening (fig. 10) and of the reaction forces (fig. 11) decreasing responses are apparent at least after 1 s of starting with the dynamic excitation. At the starting point 1 s dead load is active with 100 %. Because of the non-linear calculation the excitation input for the dynamic loading was multiplied by the safety factor 1.1 requested for service level D. The tank system remains stable with great reserves against overturning and sliding. The rag bolts are able to transmit the global shear forces of maximum 25 kN.

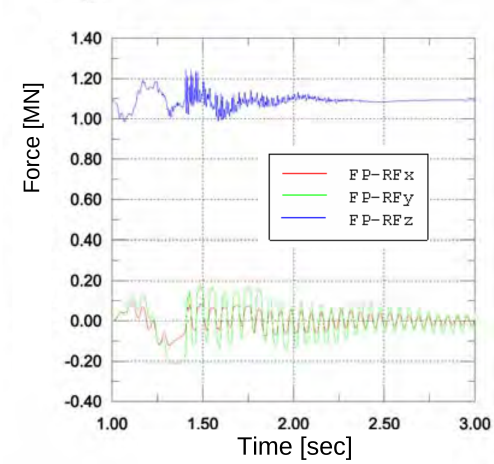


Fig. 11: Reaction forces due to EE

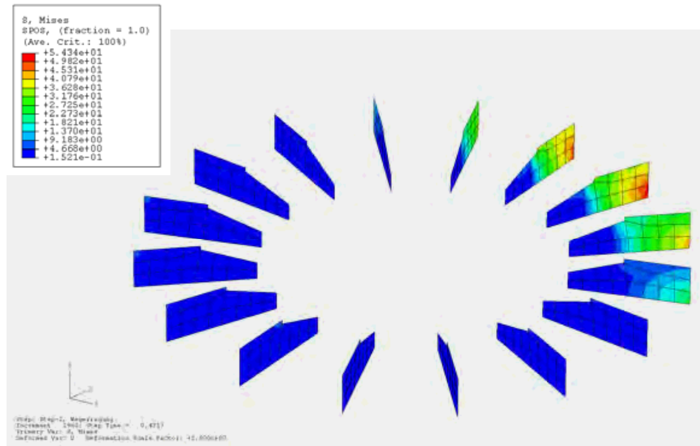


Fig. 12: Stress distribution [MPa] in support structure

Unlike due to the loading from RPP with a peak force of 917 kN the shear force response exceeds the yield limit of the rag bolts which is 430 kN for the ten bolts altogether. Therefore, the combined effect of friction and bearing capacity of the rag bolts is decisive. According to the best estimate modelling of this behaviour as described before and with the realistic shock like loading increased by the safety factor of 1.1 the tank system remains stable too. There is no overturning. But plastic deformations of the rag bolts do appear. The most important results are the reaction forces at the concrete plate (fig. 13), the displacement of the tank system in the direction of the action force (x-direction, fig. 14) and the characteristic of the non-linear spring for the rag bolts which is reproduced in the response analysis (fig. 15). The maximum bolt force reaches 80 % of the ultimate limit. It occurs after 0.1 s starting the dynamic calculation and corresponds with a displacement of 4.2 mm.

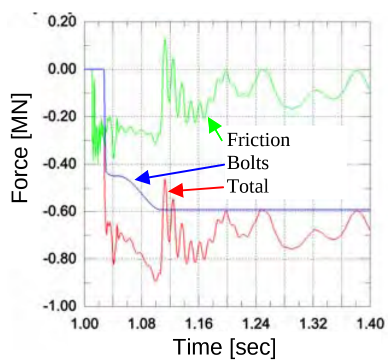


Fig. 13: Reaction forces - RPP

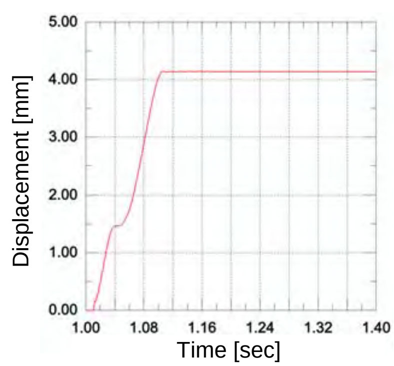


Fig. 14: Displacement of tank - RPP

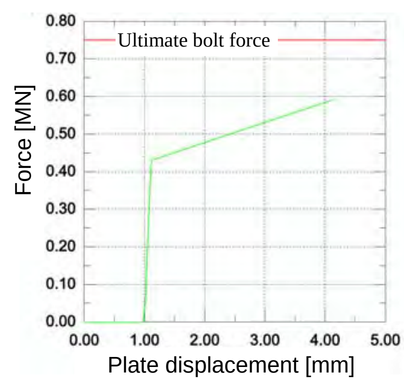


Fig. 15: Resultant bolt force - RPP

CONCLUSIONS

If under the impact of special loads with significant horizontal portions proof of stability of tall structures is not possible with simple static methods linear dynamic analyses may be appropriate. But for high load levels non-linear analyses with best estimate modelling of the non-linear effects are the only means for proof by simulation. The expenditure of analysis, on the other hand, saves extensive cost that might be spent for measures like additional supports or backfitting of the existing supports. The presented examples show the used computer program [1] to be a powerful instrument for geometric and/or material non-linear analyses. Especially combined elastic and rigid body motion analysis works well, provided that the parameters for justification the analyses are chosen reasonably.

NOMENCLATURE

BW	blast wave	A1-Boden	Acceleration direction 1 – bottom
EE	external event	FP-ux	Foot Point, displacement x-direction
FE	finite element	FP-dx	Foot Point, displacement x-direction
NPP	nuclear power plant	FP-RFx	Foot Point, reaction force x-direction
RPP	rupture of pressure pipe	U1	Displacement direction 1

REFERENCES

[1] Hibbit, Karlsson & Sorensen, Inc. (HKS), Pawtucket, Rhode Island, USA
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